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TIMBER ENGINEERING FOR DEVELOPING COUNTRIES

Part 5

Applications and Constructions *

Prepared by

Agro-industries Branch,
Division of Industrial Operations

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PREFACE

The United Nations Industrial Development Organization (UNIDO) was established in 1967 to assist developing countries in their efforts towards industrialization. Wood is a virtually universal material which is familiar to people world-wide, whether grown in their country or not. Wood is used for a great variety of purposes but principally for construction, furniture, packaging and other specialized uses such as transmission poles, railway sleepers, matches and household woodenware. UNIDO has the responsibility within the United Nations' system for assisting in the development of secondary woodworking industries, and has done so since its inception, at national, regional and interregional levels through projects both large and small. UNIDO also assists through the preparation of a range of manuals dealing with specific topics of widespread interest which are common to most countries' woodworking sectors.[1/]

The lectures comprising this set of documents are part of UNIDO's continuing efforts to help engineers and specifiers appreciate the role that wood can play as a structural material. Part 5 consists of 12 out of the 36 lectures prepared for the Timber Engineering Workshop (TEW) held 2 - 20 May 1983 in Melbourne, Australia. The TEW was organized by UNIDO with the co-operation of the Commonwealth Scientific and Industrial Research Organization (CSIRO) and funded by a contribution made under the Australian Government's aid vote to the United Nations Industrial Development Fund. Administrative support was provided by the Australian Government's Department of Industry and Commerce. The remaining lectures are reproduced as Parts 1 to 4 covering a wide range of subjects, including case studies, as shown in the list of contents.

[1/] A fuller summary of these activities is available in a brochure entitled "UNIDO for Industrialization, Wood Processing and Wood Products", PI/78.

These lectures were complemented by site and factory visits, discussion sessions and assignment work done in small groups by the participants following the pattern used in other specialized technical training courses in this sector - notably in furniture and joinery production[1] and on criteria for the selection of woodworking machinery[2].

It is hoped that publication of these lectures will contribute to greater use of timber as a structural material to help satisfy the tremendous need for buildings: domestic, agricultural, industrial and commercial as well as for particular structures, such as bridges, in the developing countries. It is also hoped that this material will be of use to teachers in training institutes as well as to engineers and architects in both public and private practice.

Readers should note that examples cited are often of Australian conditions and may not be wholly applicable to developing countries despite the widespread use of the Australian timber stress grading and strength grouping systems and the range of conditions encountered in the Australian subcontinent. Readers should also note that the lectures were usually accompanied by slides and other visual aids, together with informal comments by the lecturer, for added depth of coverage.

The views expressed are those of the individual authors and do not necessarily reflect the views of UNIDO.

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[1] Lectures reproduced as ID/108/Rev.1.

[2] Lectures reproduced as ID/247/Rev.1.

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INTRODUCTION

Many developing countries are fortunate in having good resources of timber but virtually all countries make considerable use of wood and wood products, whether home-grown or imported, for housing and other buildings in both structural and non-structural applications, as well as for furniture and cabinet work and specialized uses. It is a familiar material, but one that is all too often misunderstood or not fully appreciated since wood exists in a great variety of types and qualities.

There are certain well-known species that almost everyone knows of, such as teak, oak and pine, while some such as beech, eucalyptus, acacia, mahogany and rosewood are known primarily in certain regions. Others have been introduced to widespread use more recently, notably the merantis, lauans and keruing from Southeast Asia. Plantations also provide an increasing volume of wood. Very many more species exist and are known locally and usually used to good purpose by those in the business.

The use of timber for construction is not new and, in fact, has a very long tradition. This tradition has unfortunately given way in many countries to the use of other materials whose large industries have successfully supported the development of design information and teaching of engineering design methods for their materials - notably concrete, steel and brick. This has not been so much the case for timber despite considerable efforts by certain research and development institutions in countries where timber and timber-framed construction has maintained a strong position. Usually their building methods are based on the use of only a few well-known coniferous (softwood) species and a limited number of standard sizes and grades. Ample design aids exist and relatively few problems are encountered by the very many builders involved.

Recently, computer-aided design has been developed along with factory-made components and fully prefabricated houses with the accompanying improvement in quality control and decreased risk of site problems. Other modern timber engineering developments have enabled timber to be used with increasing confidence for an ever wider range of structures. This has been especially so in North America, Western Europe, Australia and New Zealand.

UNIDO feels that an important means of transferring this technology is through the organization of specialized training courses aimed at introducing engineers, architects and specifiers to the subject and especially drawing to their attention the advantages of wood (as well as disadvantages and potential problem areas) and reference sources so that for particular projects or structures, wood may be fairly considered in competition with other materials and used when appropriate. Cost comparisons, aesthetic and traditional considerations must naturally be made in the context of each country and project but it is hoped that the publication of these lectures will lead those involved to a rational approach to the use of wood in construction and remove some of the misunderstandings and misapprehensions all too often associated with this ancient yet modern material.

Material in this publication may be freely quoted or reprinted, but acknowledgement is requested together with two copies of the publication containing the quotation or reprint.

SPECIFICATION OF TIMBER FOR STRUCTURAL USE

William G. Keating[1]

INTRODUCTION

The value of timber as a structural material has been verified by centuries of use. However, it could have established its reputation more economically if the relationship between quality and usefulness had been more completely understood. Nowadays it is essential that costs, both erection and maintenance, are contained within competitive limits which means,

(a) keeping material quantities to the minimum necessary;
(b) designing for adequate safety and performance; and
(c) avoiding costly maintenance or premature replacement.

Another concern for the community as a whole is the depletion of the world's finite resources. In this regard, timber has a definite advantage over other materials because of its renewable nature. Still this is no excuse for wasteful practices.

Obviously correct specification is one way of overcoming the above problems. To assist in this approach most countries have established national sets of standards to lay down recommended methods of specifying materials and their correct use. Because of its importance, timber and its derived products are usually well catered for but this does not necessarily mean that such standards are always used or even well understood.

The development of standards generally, and in particular those relating to timber, has been a product of improved communications and closer international co-operation. Timber standards in the past were often prepared, written and promulgated by people with an eye to the selling of the local product without sufficient attention to the long-term reputation of a vital raw material. Gradually standards around the world have tended to become themselves standardised. This approach has succeeded to a much greater degree than many thought possible even a few years ago. It is immediately obvious that through these developments the technology transfer process has been enhanced considerably and any country, particularly a developing one, is able to take advantage of advances made in another. Engineers who saw this happening in the 'newer' materials such as steel and concrete, looked for the same with timber and depending on the particular situation, either exerted their influence on the timber standards committees or turned to the other materials.

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[1] An officer of CSIRO, Division of Chemical and Wood Technology, Melbourne, Australia.

To the specifier, standards are particularly useful, even essential, but they may need to be added to or substracted from to make a complete specification. For timber this implies an understanding of its characteristics particularly its limitations as well as its special properties.

The following is an example of specification writing that was common some years ago and shows the completely unrealistic approach often present,

> 'Scantling shall be the best of its kind cut from mature, hill-grown trees felled during the winter. It shall be perfectly straight and out of winding, die-square, bone dry and free from heart, sap and all defects.'

Modern day standards more closely match the needs of the user with the availability of the material.

BASIC WORKING STRESSES

Previous papers have shown how timber may be graded for structural purposes and how a basic working stress may be derived. As this is the figure from which the engineer starts his design process, it follows that the first requirement he specifies is the stress grade of the timber he wishes to use. In Australia stress grade is defined as - 'the classification of a piece of timber for structural purposes by means of either visual or mechanical grading to indicate the basic working stresses and stiffnesses to be used for structural design purposes. The stress grade is designated in a form such as "F7" which indicates that for such a grade of material the basic working stress in bending is approximately 7 MPa.'

TIMBER IDENTIFICATION

In many situations the engineer, provided he is satisfied as to the stress grade, is not too concerned as to the identity of the timber. However, there are also many cases where positive species identification, at least as far as a group classification, is essential. One such situation applies when there exists a biological hazard. Here, in order to remove the hazard, the engineer may need to specify one or more of the following:

(a) The use of timbers treated with preservatives. The following are three possible approaches:

  (i) the timber is surrounded by a complete envelope of preserved timber of such depth that provides for post treatment checking and minor fabrication damage and is carried out after all cutting, drilling, notching, machining, etc., has been done;

  (ii) the timber is treated through the full depth with adequate retentions of preservative; or

  (iii) the susceptible sapwood only is treated against Lyctus attack.

(b) The use of naturally durable heartwood species which can be identified on site, for protection against fungal attack only.

(c) The provision of adequate ventilation against fungal (decay) attack.

(d) The use of soil poisoning and/or mechanical barriers such as 'ant' capping against termites.

In (a) and (b) above, species identification is essential.

For the design of timber structures, it is often necessary to use unseasoned timber as the member cross-sections are so large that it may be uneconomic to specify that they should be dry. This is not necessarily a problem as construction techniques are available to minimize the effects of shrinkage but again species or species group identification is important in order to predict the likely performance. The use of hardwoods tends to accentuate the situation for, as compared to softwoods, their total volumetric shrinkage is usually much higher and the differential percentage shrinkage in the tangential and radial directions is also higher. In this field it may also be of some importance to know the identity of those pieces that could be subject to the abnormal form of shrinkage known as collapse. Standard texts will list those species that are known to be susceptible, but identification is necessary.

DIMENSIONS AND TOLERANCES

It is important that there is no cause for ambiguity when stating dimensions and tolerances. In Australia lineal metres (m) or cubic metres are the units of measurement for length and volume respectively and the cross-sections are referred to by millimetres (mm). The convention that is being encouraged is to specify cross-section, number of pieces and lengths in that order, e.g. 100 x 50 60/2.4, where 100 x 50 indicates the required cross-section in millimetres and 60/2.4 indicates 60 pieces each 2.4 metres in length. For tolerances, the designer should make it clear to the supplier, usually by quoting the appropriate standard, that he understands for example, that a negative tolerance on cross-section is permissible. Usually this applies only to unseasoned sawn timber. For dressed or seasoned timber, there is normally no negative tolerance. These points and the conditions relating to length tolerances are usually spelt out in the standard.

STANDARDS

The value of standards becomes obvious when ordering timber. By quoting the stress grade, the identity of the species (to the precision necessary) and also quoting the number of the standard to which the timber is produced, a considerable amount of information is conveyed in a very small space. When that is combined with, say, a timber engineering code, the intentions of the designer and the responsibilities of the supplier are quite clear.

If no standard exists, it is normal practice to examine standards from other countries and modify them where necessary to suit the local conditions. As can be imagined this is not very satisfactory. If even this is not possible and trained graders are not available, some simple rules combined with a minimum density clause may be satisfactory. Such an approach would need to be conservative but it could allow timber to be used.

EQUILIBRIUM MOISTURE CONTENT

When structures or elements are to be fabricated with seasoned timber, the designer should ascertain the average equilibrium moisture content for the environment in which the structures or elements are to be erected. He should then specify that the timber used shall have a moisture content at the time of fabrication within 3 per cent of this average value.

Wood exposed to an atmosphere containing moisture in the form of water vapour will come, in time, to a steady moisture content condition, called the equilibrium moisture content. This steady moisture state depends on the relative humidity, the temperature of the surrounding air and the drying conditions which it has previously undergone. It fluctuates with changes in one or both of these atmospheric conditions. Such changes produce corresponding changes in the dimensions of wood. Obviously to minimize the extent of such movement it is desirable to install timber at a moisture content mid-way between the extremes it is likely to reach in service. Around 12 to 14% is about an average figure but could be higher in tropical areas or considerably lower in locations such as central Australia or indoors in air-conditioned buildings.

CORROSION

Corrosion may occur in metal connectors when used under moist conditions and under certain atmospheric conditions such as might be encountered in marine environments, certain factories or near chlorinated water. Condensation in roofs of heated buildings will also encourage corrosion. Some preservatives of the waterborne type may, under moist conditions, cause corrosion to unprotected metal work. The designer must ascertain if such hazardous conditions are likely to be encountered and ensure that the appropriate precautions are taken.

On the other hand, wood itself does not corrode which makes it a most useful material to specify for use in a corrosive environment.

PRESERVATIVE TYPE

It is not sufficient that the designer specifies that the timber members be preservatively treated should that be necessary. He must specifically nominate the preservative type. Oil, waterborne and light organic solvent types each have their own advantages and limitations so that in order to make the correct choice some attempt to acquire the necessary background information is required.

TRANSPORT AND ERECTION

The overstressing of timber members during transport and erection should be carefully avoided. In the case of framed arches, trusses, portal frames and the like, special care is necessary to avoid distortion in hoisting from the horizontal to the vertical position. For this reason the designer should indicate lifting points or methods of lifting on the design.

BUILT-UP MEMBERS

Often it is not possible to obtain solid timber members of sufficiently large cross-section or length to suit a particular need. This does not preclude the use of timber as large members may be formed from much smaller pieces. However, for engineered structures this is a specialised technique and one that needs a first class quality control system. For this reason the designer, should he wish to use members such as glue-laminated beams or nail laminated components, must seek guidance from an appropriate standard and order from a well-known and reputable firm.

Simple members may be fabricated on-site, but close supervision is usually necessary.

STORAGE ON SITE

Unless specifically designed for the purpose, timber components and structural elements should not be exposed to high humidity, and all materials and assemblies in storage should be protected against exposure to the weather, wetting, damage, decay and insect attack. Adequate ventilation of the stock must be provided.

INSPECTION ON SITE

It is not sufficient for the designer to just write the specification even if it does cover all the aspects mentioned previously. He must make provision for a system of on-site check inspections to be made on a regular basis. This implies that there is some one present such as the site engineer or foreman who is at least familiar with the standards quoted in the contract, any special conditions and the way in which the checks should be made.

For example, if dry timber has been specified, this would normally indicate that this is critical to the performance of the structure. In structural members if green timber is supplied in place of the specified dry, failure is possible, excessive deflection is probable and end splitting may occur rendering the jointing devices inoperative. For these reasons, it is good practice for a moisture meter to be on the site with some one available who knows how to use it. Similarly with preservative treatment and conformance to the grading rules. Branding is not always a cause for complete assurance, but it is a useful safeguard and indicates the supplier will back his product which is often why it was specified in the first place. For this reason checks are necessary to ensure that the agreed branding has been done in the correct manner and frequency.

SUMMARY

Timber is a well proven structural material but its familiarity often leads to its mis-use. Modern technology has enabled us to specify timber in much more economic (i.e. smaller) sizes over a wider range of conditions than in the past, but at the same time the need for close attention to specification detail has been markedly increased.

REFERENCES

Panshin, A.J., and de Zeeuw, G. (1970). Textbook of Wood Technology, Volume 1, 3rd edn. McGraw-Hill Book Company, New York.

Standards Association of Australia (1982). Draft revision of SAA Timber Engineering Code.

Timber Promotion Council (1982). Understanding Victorian Structural Hardwood. Melbourne, Aust.

PLYWOOD IN CONCRETE FORMWORK

Kevin J. Lyngcoln[1]

The above lecture was based upon a manual of the same title by Mr. Lyngcoln. The manual is not reproduced in this document. However, it may be obtained from the Plywood Association of Australia Ltd.[2] at a cost of $A 4.50 plus postage per copy (price in March 1984). A copy of its cover page is shown below, and a copy of its list of contents and other relevant details are included.

PLYWOOD IN
CONCRETE FORMWORK

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PLYWOOD IN CONCRETE FORMWORK

This manual has been produced for the construction industry by the Australian Plywood Promotion Council in association with the Plywood Association of Australia.
Edited by K. J. Lyngcoln ARMIT., MIE(Aust.), AIWSc., Engineer, Plywood Association of Australia Ltd.

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The Plywood Association of Australia wishes to express its thanks and appreciation to all of the organisations and individuals who have assisted in making this publication possible by way of contribution or criticism. For their special involvement, the Association thanks Mr. Peter McAdam, Consulting Engineer, whose assistance in preparing Section 1 covering the derivation of concrete pressures and other advice on formwork construction was invaluable, and the Cement and Concrete Association of Australia for their co-operation.

FOREWORD

Concrete in Australia over the past few decades has developed into one of the most used materials in the building industry. Its ability to be moulded into a great variety of shapes, its ability to accept a wide range of surface finishes, and its inherent strength and durability, provide designers with an extremely versatile design medium. The development of "off-form" finishes in particular has given architects yet another means of creating building facades in economic and aesthetically pleasing forms.

The success of concrete as a visual material depends in no small measure, however, on the quality of the formwork that moulds it into the desired shape, and in particular on the quality of the form face. Plywood has become established as a major material for this purpose.

Production of "off-form" finishes imposes much tighter limits on concrete formwork. In particular, water absorption of the form face, joint sealing and deflection under load are critical. Whilst in most cases the structural strength of formwork systems is more than adequate, the control of deflection and the sealing of joints are sometimes not given the same attention and the concrete finish suffers accordingly.

This Manual is a welcome addition to Australian publications on the subject as it addresses itself to these problems, provides the building industry with efficient design methods and suggests effective details to ensure that the best performance is obtained from concrete. Consideration is given also, in the section on Formwork Pressure, to the effects on formwork design of modern trends which involve more workable concretes, greater rates of concrete placement and emphasis on the overall speed of construction. Each of these tends to impose additional loads and pressures on the formwork and demand more accurate designs to ensure that strength, deflection and sealing requirements are met.

Plywood has all the attributes to make it an excellent material for formwork. This Manual provides the building industry and teaching institutions with an uncomplicated, concise text describing efficient and effective methods of using it for this purpose.

K. J. Cavanagh
Director, Cement and Concrete
Association of Australia.

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TIMBER STRUCTURES - DETAILING FOR DURABILITY

Leslie D. Armstrong[1]

Extensive deterioration of building materials will occur in the absence of good principles of construction, use and maintenance of buildings and other structures. Where high rainfall, high humidity, extensive insolation and strong winds prevail, some types of decay of materials can become rapid without proper emphasis on good practice.

The weathering factors of the tropics are those operating in temperate regions, although the considerable variations in intensity make some factors insignificant and others important. Some processes of degradation, particularly attack by biological agencies, are likely to be hastened by uniform temperatures at the optimum level as experienced towards the equator, rather than by widely fluctuating values, but the reverse applies for some physical effects, such as disruption from dimensional change, which will be at a maximum in hot arid regions where low minima also occur.

Skilled designers attempt to achieve the best performance in buildings through the selection of durable materials, through the exercise of good principles of construction and workmanship and through proper treatments and maintenance to increase durability. These important approaches are never fully achieved even in large cities where materials and skilled labour are abundant and usually only partially achieved in smaller cities and towns in remote regions where materials and skilled tradesmen are in short supply.

The behaviour of timber in service is affected by environmental and biological factors which vary in their influences depending upon the conditions of exposure. Usually, the worst conditions are present under outdoor exposure because climatic variations are widest and of greatest impact, but severe conditions may also occur indoors due to hazardous, artificial environments or due to poor techniques of construction.

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[1] Formerly an officer of CSIRO, Division of Building Research, Melbourne, Australia.

TIMBER USED INDOORS

The major problems of deterioration of structural timbers used indoors are usually associated with excessive amounts of liquid water in direct contact with the timber, excessive amounts of water vapour in the atmosphere, excessive fluctuations in the moisture content of the wood, extremes of temperature, or chemical fumes. When the moisture content of wood exceeds about 25 per cent, based on oven-dry weight, biological decay, due to bacteria or fungi, may occur. The effects intensify with elevation of temperature. Should the moisture content of the wood fluctuate appreciably with changes in atmospheric conditions (which may occur in industrial buildings), excessive amounts of swelling and shrinkage may arise which can cause severe stressing and degrade of the timber and associated joints. Checking or splitting of the wood surface may also occur, with the subsequent development of bacteria or fungi in the checks and associated decay of the wood. In tropical climates, high values of relative humidity are maintained for extensive periods and in order to inhibit biological attack in timber there is usually a strong need to apply preservative chemicals and protective films to the surfaces of members and to ensure adequate ventilation of air spaces. In addition, a high hazard exists from water penetration and flooding of structures, which requires proper shielding and drainage systems to shed water and prevent retention.

Although timber in service is rarely affected by chemical fumes, the metal fasteners commonly used in timber joints may corrode unless they have been treated to withstand specific chemicals.

TIMBER USED OUTDOORS

When used externally, timber may be partially or fully exposed to the weather and its moisture content may reach undesirable values or may fluctuate widely due to exposure to the sun, rain, snow or wind. In some cases harmful chemicals may contact the timber and joints. At the planning stage, building orientation should be considered to reduce exposure of vulnerable components. Protective treatments consisting of durable surface coatings or penetrating chemicals may have to be used to help the timber to resist damaging agents or to inhibit moisture changes. These treatments assist in keeping swelling stresses to a minimum and reduce the incidence of biological decay. Consideration should be given to the method of assembly of timber components so that, as far as possible, water cannot pond on surfaces nor enter the joints and any water that does intrude can drain away. Moisture should also be prevented from diffusing into wood from porous media (such as soil or concrete) by avoiding direct contact. As well as applying these initial treatments, continued observation and maintenance will be needed during the life of the structure, as is the case with all structures and materials.

The methods of providing physical protection for timber structures and components fall into two categories namely, shielding and structural detailing.

SHIELDING METHODS

Structural timber exposed on the outside of buildings may be shielded from the weather by utilising natural and/or artificial systems of protection. By carefully selecting the type and location of trees and shrubs, timber walls and exposed structural timbers may be substantially protected from the wind and rain to inhibit the ingress of water and reduce excessive fluctuations in moisture content. Shielding from the sun is also provided with shrubs, not only giving protection to exposed timber but also assisting the internal environment of a building through shading of walls and windows. Careful selection of the species of trees and shrubs is essential so that an acceptable degree of shielding is presented to the building.

The trees and shrubs must be of sufficient distance from the building and of adequate spacing so that the foliage does not provide a wet source nor inhibit the flow of air adjacent to the walls, otherwise the moisture content of exposed timber may reach detrimental levels. The species of trees must be selected so that no danger is presented to the buildings during windstorms. Further, the root growth and moisture requirements of the trees must also be considered so that no problems will arise with the foundations of buildings due to excessive moisture changes in the soil or intrusion of roots into the structure. Consultation with botanists and experienced nurserymen is essential during tree selection, planting and landscaping.

These examples illustrate various methods of protecting timber structures:-

1. Shielding

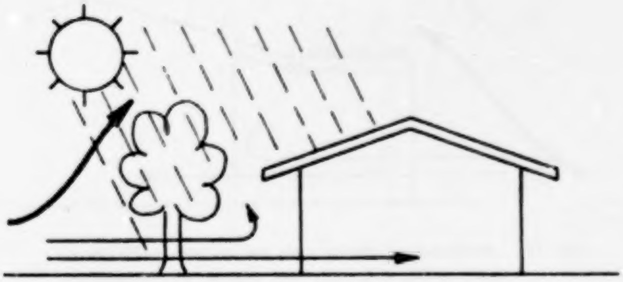


Fig. 1a **Vegetation** - intercepts sun, wind and rain, but allows air circulation near the building to inhibit moisture build-up.

Where it is not convenient to use trees for shielding, artificial systems may be installed to achieve a similar purpose. Roof overhangs, shading devices, and screen walls or fences can be constructed of decorative and durable materials to provide protection against the elements, to assist indoor temperature control and to enhance the architecture. Practical experience throughout the world has demonstrated the extremely valuable contribution made by shielding systems in prolonging the life of exposed building materials.

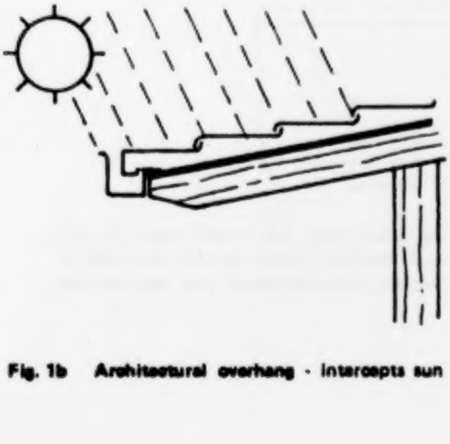


Fig. 1b **Architectural overhang** - intercepts sun and rain.

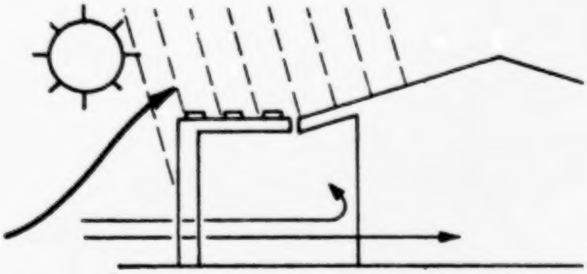
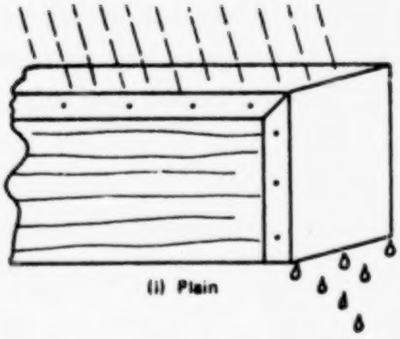
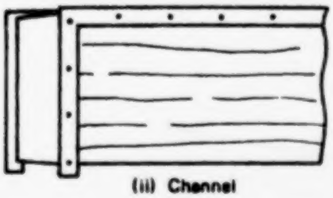


Fig. 1c Architectural screen wall and pergola - Intercepts sun, wind and rain but allows air circulation near the building.



(i) Plain



(ii) Channel

Fig. 1d Flashing - sheds water and protects upper and end surfaces of the beam. Shield does not fold under to lower surface, otherwise water may become trapped.

Other methods of shielding consist of applying caps or flashing directly to the surfaces of exposed timber members to prevent water lodging on surfaces and to divert the water away from the members.

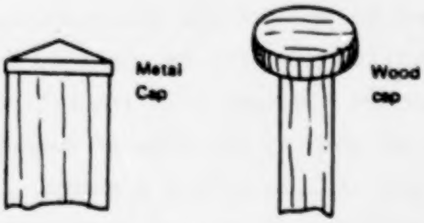


Fig. 1e Caps - protect end grain.

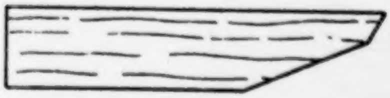


Fig. 1f Shaping - encourages water run-off and decreases exposure of end grain to sun and rain.

Improperly designed shielding systems may accentuate rather than alleviate degradation of timber by trapping water that has leaked into the shields. A shield or cap should only cover the exposed surface that requires protection, leaving adequate ventilation and drainage of the remaining surfaces, especially the lower surfaces, so that moisture will not be retained on the timber to cause prolonged wetting and subsequent biological decay.

The end-grain surfaces of wood need special treatment with respect to shielding and sealing. Moisture is gained and lost more readily through end-grain than through side grain surfaces. Checking of end-grain surfaces may become severe because of differences in swelling in the tangential and radial directions (relative to the growth rings) and biological decay may develop in the open checks. Shields of the types already described may be formed to protect end-grain surfaces and a variety of sealing paints and films can also be applied. Further beneficial treatments that have proved successful in inhibiting the deterioration of end-grain consist of shaping the ends of members to reduce exposure and encourage water run off, machining a groove into the face or attaching a strip to provide a drip bead, or sub-dividing the large end face of a member into a number of small sections by means of deep saw kerfs in order to relieve swelling stresses and reduce checking.

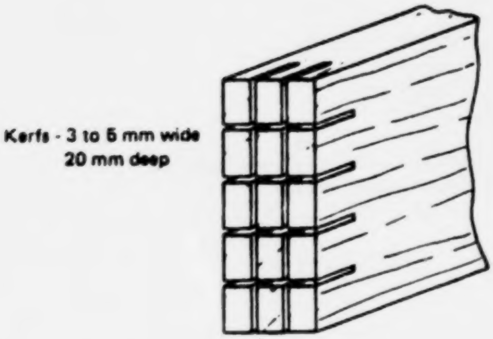


Fig. 1g Saw kerfs - relieve swelling stresses and reduce checking.

Timber that is totally immersed in water will not decay. However, when a structural member, such as a pile, is partially immersed in water and partially exposed to air, degradation of the wood may occur at the waterline unless protective methods are applied to reduce moisture content fluctuations and to inhibit checking and subsequent decay. One method of treatment is to apply a bandage containing preservative to the section at the waterline.

STRUCTURAL DETAILING

When timber components are joined together or brought together into contact with other structural members or materials, care must be taken to avoid the entry of water and, more importantly, to avoid trapping water in the joint. Shaping, caulking and flashing of timber members can encourage drainage of water away from joints and also inhibit the entry of water. Grooving and drilling the faces of members can encourage drainage of water from inside joints where leakage may be present. The transfer of moisture from another porous medium (e.g. soil or concrete), to timber should be prevented and the design of the joint should allow easy access for the application of maintenance treatments, especially to end-grain surfaces. Where metal shoes or fittings are used to support the ends of members they should not enclose the member or encourage trapping of water. If initially green timbers are used in construction, the joints, fasteners and support fittings must be selected to accommodate the subsequent shrinkage of the timber and permit maintenance. Similarly, where excessive moisture content fluctuations are expected, provision must be made for the inevitable dimensional changes that will result. Several examples of good structural detailing are shown in the figures.

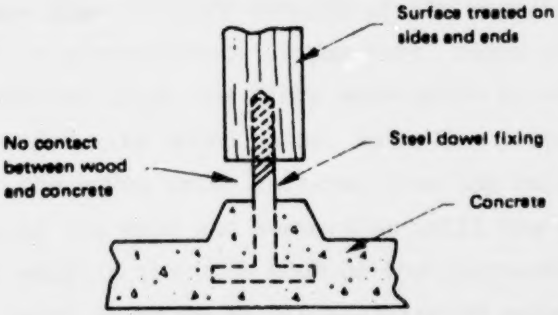

Surface treated on sides and ends
Steel dowel fixing
No contact between wood and concrete
Concrete

Fig. 2a Isolation of wood column from wet source using a steel dowel

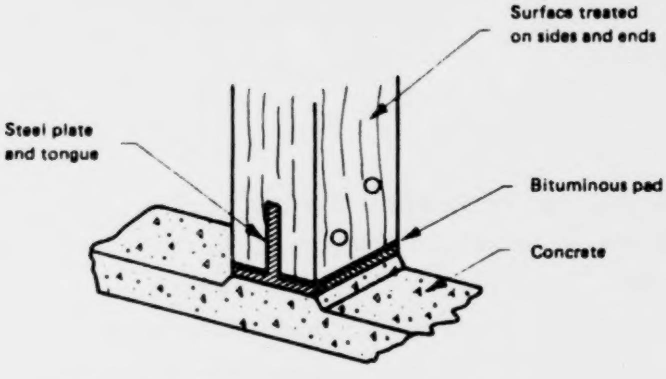

Surface treated
on sides and ends
Steel plate
and tongue
Bituminous pad
Concrete

Fig. 2b Isolation of wood column using a steel plate and a bituminous pad

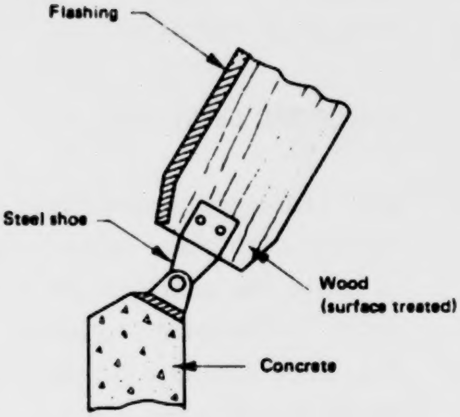

Flashing
Steel shoe
Wood
(surface treated)
Concrete

Fig. 2c Isolation and shielding

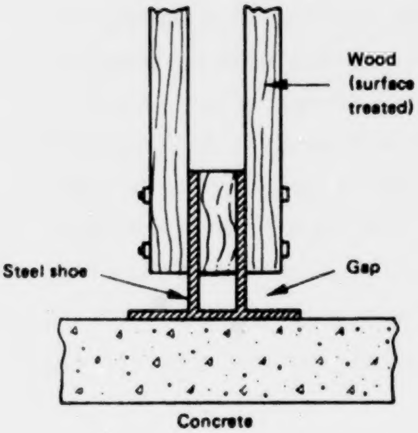

Wood
(surface
treated)
Steel shoe
Gap
Concrete

Fig. 2d Support of a spaced column

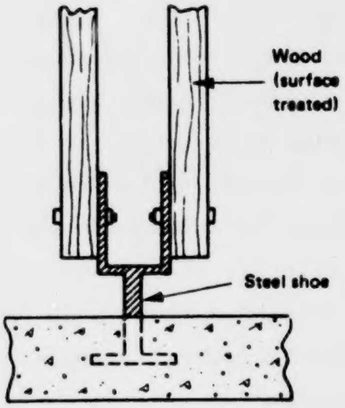

Wood
(surface
treated)
Steel shoe

Fig. 2e Alternative method to 2d

USE OF GREEN TIMBER IN STRUCTURES

Leslie D. Armstrong[1/]

Wood is a hygroscopic material that gains and loses water with variations in the moisture content of the environment in which it is used. Because diffusion of moisture in wood is a rate process there is a time lag in the variation of the moisture content of wood with respect to changes in atmospheric conditions. Moisture gradients develop below the surface of the wood and severe distortions may occur in some timber components.

In service, timber may be fully immersed in water with the result that its moisture content remains constantly high, or it may be exposed to a variety of natural, controlled or widely variable artificial environments in which the moisture contents remain steady or fluctuate over a considerable range. The moisture content of wood gradually varies in accordance with the surrounding conditions and, given sufficient time, it will equilibrate with a particular environment, i.e. the moisture content of the wood will reach a steady value which will be related to the moisture content of the surrounding air. In most countries, distinct moisture content levels occur in timber corresponding with the climatic seasons. In Melbourne, Australia, the moisture content of wood used under shelter varies from about 10 to 15 per cent, based on the oven-dry weight of the wood, over each year.

Wood in the tree contains water in two forms, namely free water contained in the hollow lumen of the wood cell and chemically bound water contained in the wood tissue in the cell wall. As wood dries, the free water is first removed from the cell lumen to the surface of the wood where it evaporates. The moisture content of the wood following removal of the free water is approximately 30 per cent, based on oven-dry weight, and is referred to as the fibre saturation point, as the cell wall remains saturated with bound water. As drying proceeds below this moisture content, the bound water diffuses from the cell wall to the external surfaces of the wood and evaporates until the moisture content of the wood is in equilibrium with that of the surrounding air. This moisture content level is known as the equilibrium moisture content of the sample, abbreviated to 'e.m.c.'. In Melbourne, the e.m.c. ranges

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from about 10 to 15 per cent in timber exposed to ambient conditions and protected from direct rain and sun.

During loss of free water from the cell lumens of wood there is little effect on the physical properties, except that in some species of hardwood a severe form of abnormal lateral shrinkage, known as 'collapse', can occur and cause extensive distortions and dimensional changes in the timber. Changes in cross-section of up to about 8 per cent of the original dimensions can occur with cell wall collapse in some species. Precautions must be exercised in using such timbers in construction.

With loss of bound water from the cell wall during drying of timber below the fibre saturation point, the mechanical and physical properties of wood are considerably affected. An important phenomenon that accompanies moisture change below f.s.p. is that of shrinkage and swelling of the lateral dimensions of wood with desorption and adsorption of moisture. Shrinkage parallel or tangential to the growth rings is much larger than that in the radial direction. It is often as much as 2 to 1. Shrinkage along the grain of wood is of little or no practical significance. In softwoods, depending on species, shrinkage of up to about 6 per cent of the original lateral dimensions can occur during drying from the fibre saturation point down to a typical air-dry condition of 12 per cent moisture content. In hardwoods, shrinkage of up to 10 per cent of the lateral dimensions may occur under similar conditions.

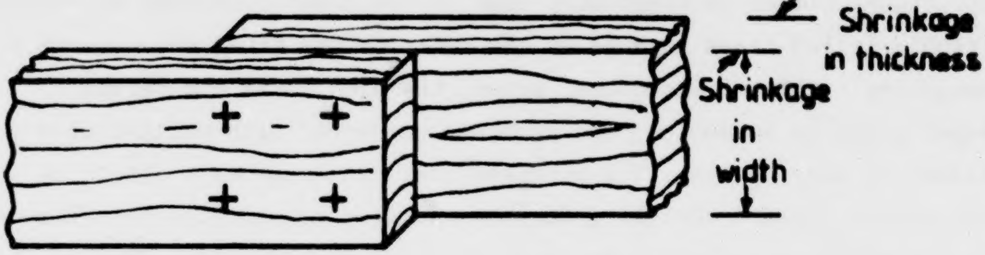
Apart from making allowances in some cases for the shrinkage and swelling, or 'working', of dry timber during limited amounts of moisture content fluctuation with climatic changes, the major changes in dimension that accompany drying of initially green timber in service need special attention. This is particularly important with joints in timber components.

Obviously furniture and joinery timbers should be dried to a moisture content equal to the mean of the range anticipated in practice, whether exposed to natural ambient conditions or artificially generated conditions. Structural timbers installed in the dry state should be treated in a similar manner. Timbers used over a short duration in extreme conditions, e.g. formwork, may need surface treatment and protection to inhibit moisture changes and associated dimension changes.

Structural timber is frequently used in the green condition as cut from freshly felled trees because of the difficulties often encountered in seasoning timber of structural sizes, the high costs and delays experienced in seasoning, and problems concerned with nailing of dry timber of many species. The problems that can arise as a result of structural timbers shrinking during extensive moisture content change are many and varied and sometimes give timber a bad name. Usually, however, any such troubles caused by shrinkage are the results of thoughtlessness or carelessness on the part of the designer or builder in failing to make provision for the well established shrinkage behaviour of construction timbers. Some of the common problems encountered in practice and the precautions needed in construction methods to reduce the detrimental effects of shrinkage will be described.

When initially green structural timbers of similar shrinkage characteristics are lapped or spliced using nails, bolts or connector rings and the direction of the wood fibres (grain of the wood) is disposed in the same direction in all timber members, as shown in Figure 1, no restraints or splitting of the wood can occur between parallel rows of fasteners. However, bolt holes in the green timber must be drilled 1 to 2 mm oversize, depending upon the species and bolt diameter, otherwise splitting will occur in the grain direction adjacent to holes as the hole diameter reduces across the grain with shrinkage of the timber. In hardwoods, tangential shrinkage may amount to 15 per cent during drying from the green to the air dry state and in softwoods the shrinkage may reach 6 per cent. Bolts need re-tightening at intervals as the timber shrinks in thickness and it is essential to use the correct size of washers under bolt heads and nuts to avoid crushing of the timber and to keep bearing stresses within safe limits.

When the shrinkage characteristics of lapped or spliced green timbers are widely different or when the relative directions of the grain in adjacent members may cause significantly different shrinkage effects across a joint, staggering of fasteners in adjacent rows, as shown in Figure 2, will often alleviate splitting during shrinkage.


Shrinkage in thickness
Shrinkage in width

No restraints occur between fasteners in timbers of similar shrinkage properties and of similar grain direction.
For bolts, drill holes 1-2mm oversize.

Figure 1 Shrinkage in a timber joint

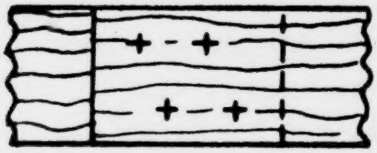


Figure 2a Staggered fasteners joining green timbers of different shrinkage properties

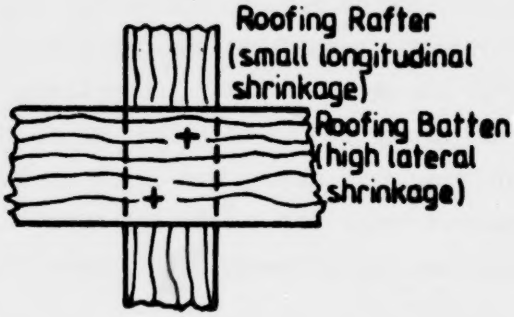

Roofing Rafter (small longitudinal shrinkage)
Roofing Batten (high lateral shrinkage)

Figure 2b Staggered fasteners joining green timbers of different grain direction

Steel plates are often used to splice green timber members butted together at their ends. When the joint is formed using a single row of bolts or screws along the length of the members, as shown in Figure 3a, no restraints occur across the joint provided bolt holes are drilled oversize in the timber. However, when two or more rows of fasteners are used in such an assembly, as shown in Figure 3b, alternate rows of holes in the steel plates must be slotted laterally to allow for displacement of the bolts or screws across the splice plates with shrinkage of the green timber members. Similar treatment may be needed when metal straps or angle brackets are used adjacent to green timber members (Figure 3c). Bolts or screws should be tightened regularly while shrinkage of the timber is in progress.

Built up timber beams and trusses are often constructed of green hardwood members and webs or gussets of plywood, hardboard or galvanized iron. Common practice is to use two or more rows of nails, staples or rivets, as far apart as possible. This is satisfactory for small beam flanges or truss members but not for members 10 cm or more in depth as the green members shrink and the gussets do not. As a result the gusset is damaged, the fasteners are overstrained or the timber members are split. Correct practice is to use the minimum number of rows of nails or rivets, to space the rows as close together as permissible, to group the fasteners along the centre lines of the members and to stagger the fasteners in adjacent rows (Figure 4).

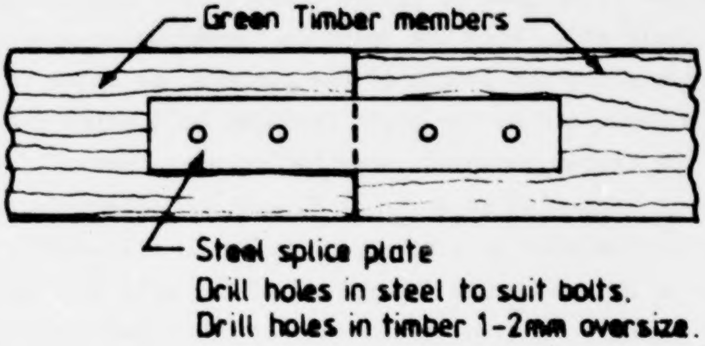

Green Timber members
Steel splice plate
Drill holes in steel to suit bolts.
Drill holes in timber 1-2mm oversize.

Figure 3a Timber/steel plate assembly with single row of bolts

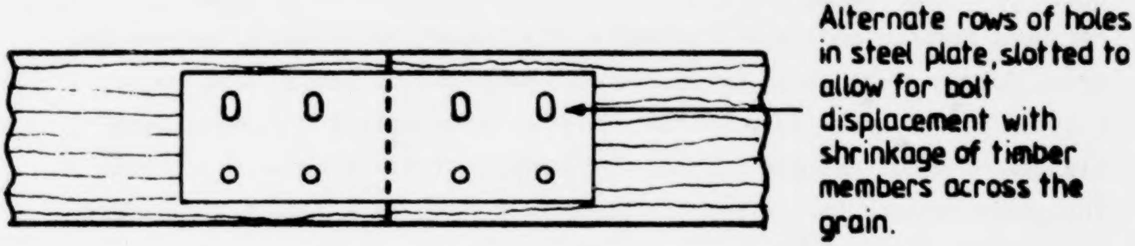

Alternate rows of holes in steel plate, slotted to allow for bolt displacement with shrinkage of timber members across the grain.

Figure 3b Timber/steel plate assembly with two or more rows of bolts or screws

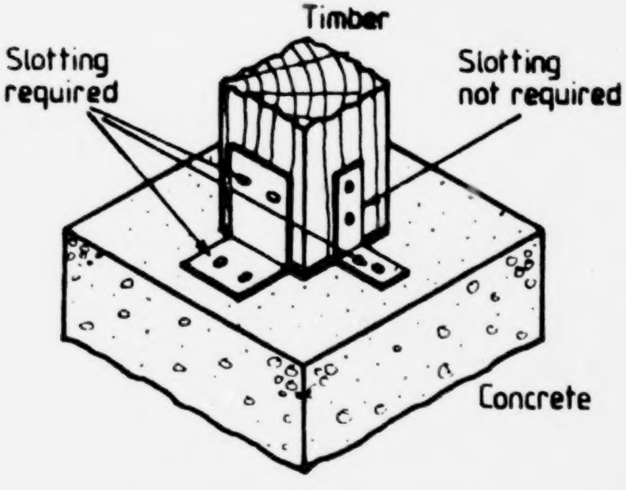

Timber
Slotting required
Slotting not required
Concrete

Figure 3c Slotted holes should be provided in metal fittings to allow for shrinkage of adjacent timber

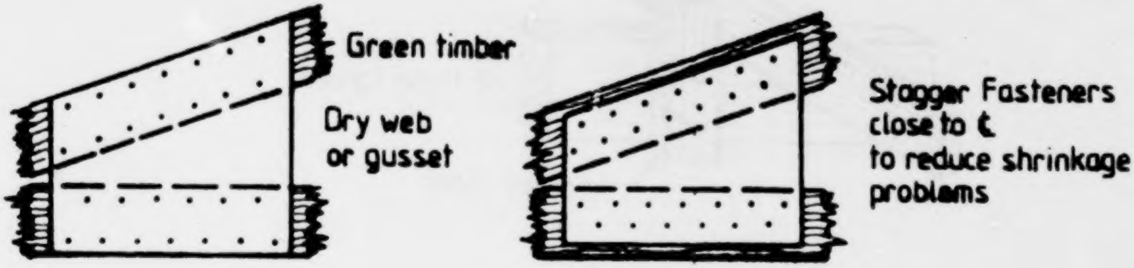


Figure 4 Reducing shrinkage problems in built-up components

When green hardwood purlins are attached to wooden cleats as shown in Figure 5, shrinkage trouble will occur in two places. The purlins and upper chords will shrink in their depth whereas the cleats will not shrink in their length. When the purlins shrink, they will no longer rest on the top chord of the truss but will hang from the bolts attaching them to the cleats. When the top chord shrinks, the two fastenings, either bolts or nails will be forced closer together and this may cause the cleat to split or break the joint. Either way, the cleat is likely to become ineffective. Three more satisfactory purlin fixtures are shown in Figure 6.

The nails or bolts attaching the purlin should be placed as close as practicable to the lower side of the purlin. If two bolts or nails are needed at each point of fixing of the purlin, these should not be placed in line across the depth of the purlin, as this will probably cause it to split. Figure 6 shows a simple strap hanger for the purlin.

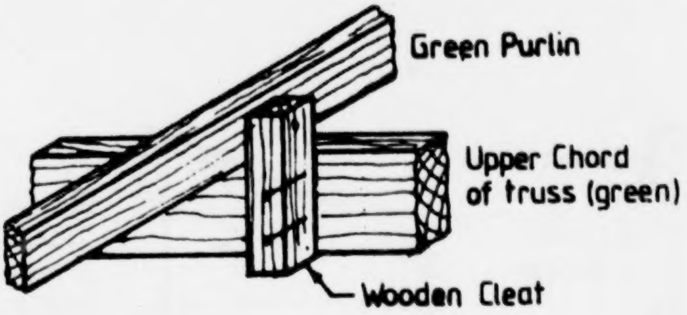

Green Purlin
Upper Chord of truss (green)
Wooden Cleat

Figure 5 Poor fixing of green timber members

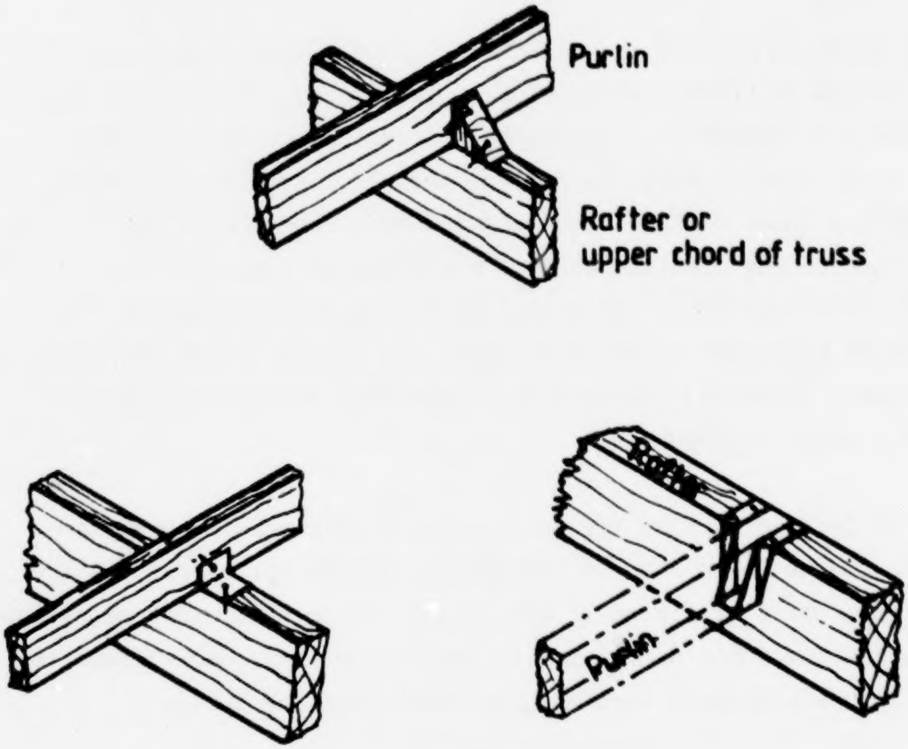

Purlin
Rafter or upper chord of truss
Rafter
Purlin

Figure 6 Improved methods of fixing green timber members

Floors, decks and verandahs are sometimes constructed of green timber joists which are supported on brickwork at one end and a green timber beam at the other end, the beam, in turn, being supported on steel or timber posts, as shown in Figure 7a. Shrinkage in the depth of the green timber beam may exceed 25 mm in a typical case using hardwood, causing this amount of misalignment in the floor. The problem may be solved by using similar support beams at each end of the floor framing so that equal changes in height occur with shrinkage of the beams (Figure 7b).

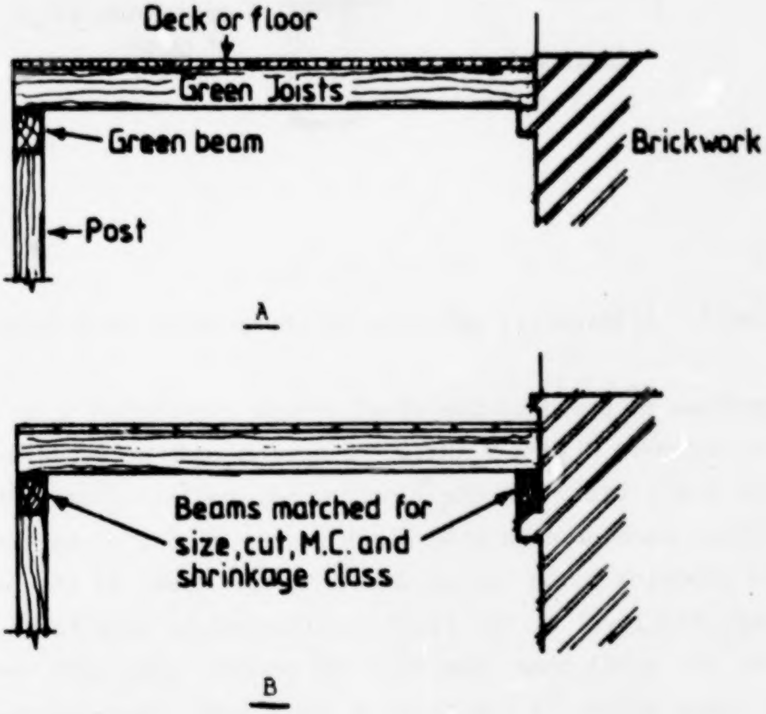


Figure 7 Prevention of differential settlement in a floor

The solution given to the problem illustrated in Figure 7 involves the additional expense of providing a second beam. A cheaper alternative is shown in Figure 8. The joists are butted against the inner face of the support beam and rest on a 75 x 35 mm ledger (or light brackets) nailed or bolted near the lower edge of the beam. The shrinkage of the ledger will only be a fraction of that of the main beam and the slight loss of level would be barely noticeable. Incidentally, square notching of the joists can cause serious loss of strength but the 1 in 3 sloping cut below the notch will obviate this.

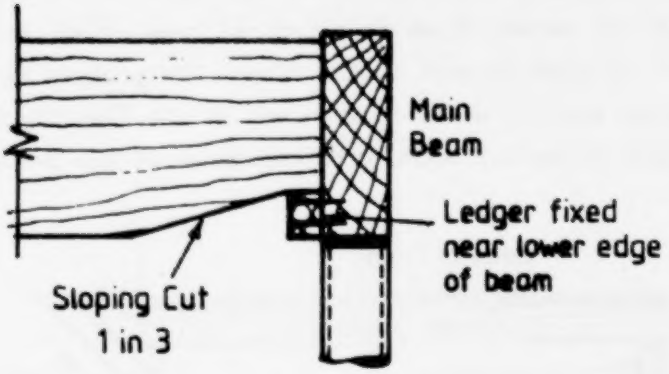


Figure 8   Alternative solution to the problem in Figure 7

A similar problem is posed by the steel I-beam and wooden joist construction shown in Figure 9. Here the joists rest on ledgers bolted to the web of the I-beam, and the flooring may be laid flush with the top of the steel beam or a packing block may be placed on the top flange to allow the flooring to be nailed down over the beam. As the hardwood joists shrink, the level of the flooring attached to them falls. However, over the steel beam, the flooring cannot move, with the result that marked humps appear in the floor at each beam. There are several ways of avoiding this trouble. In one method, a removable floor section above the steel beam may be adjusted in height by suitable packing strips to correspond with the changing level of the rest of the floor as the timber joists shrink. Another method is shown in Figure 10. Here the joists are simply notched out to take a piece of the same green hardwood and this acts as a nailing strip for the floor above it. It is, however, essential that a gap equal to at least 10 per cent of the depth of the joists be left between the under side of the nailing strip and the flange of the I-beam. Then as the joistsshrink, the floor as a whole will go down with them without interference from the steel beams.

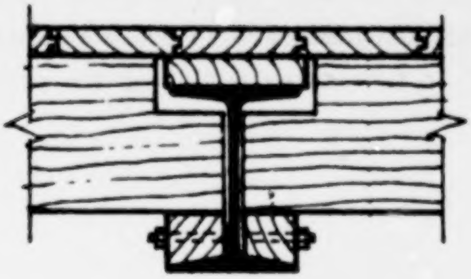


Figure 9 Shrinkage problems in timber/steel floor systems

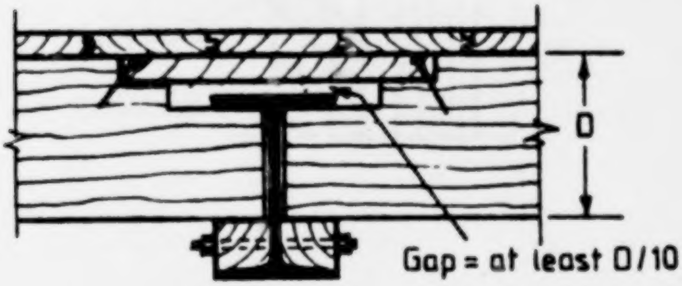


Figure 10 Solution to the problem illustrated in previous figure

In brick veneer construction in housing the structural frame is often fabricated from green timber and the outer cladding consists of fired-clay bricks to provide protection against the elements and also to impart a pleasant surface with low maintenance. The brickwork remains at a constant height but the timber frame reduces in height as the green timbers shrink on drying. The total depth of timber subject to shrinkage in the height of the frame amounts to about 300 mm, which may yield a drop in height of from 5 to 40 mm at various positions in the frame. Adequate clearance is required where window sills, soffit linings and roof rafters project from the frame over the top of brickwork. As an example, the provision of necessary clearance between the soffit and the brickwork near the top of the wall is illustrated in Figure 11. Similar precautions must be applied elsewhere to prevent contact between timber and brick components. In two-storey buildings the problems become accentuated.

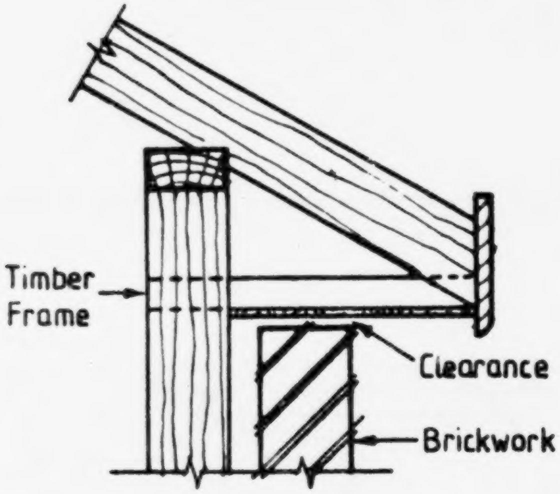


Figure 11 Provision of clearance between the soffit lining and brickwork in brick veneer construction

Metal split-ring connectors and shear plates are used in the joints of large timber structures to resist high forces. These components are adequately described in another paper. Precautions must be exercised when installing these fasteners in green timber otherwise severe splitting of the timber in contact with the fasteners can occur with shrinkage during drying. The circular groove for split rings must be of the recommended dimensions so that the gap in the ring will be of the correct clearance, when installed, to allow for anticipated shrinkage without offering restraint and possible splitting of the timber. When two or more rows of split rings are installed across intersecting green timber members having different shrinkage characteristics or grain directions, splitting between the rows can be minimized by making a saw cut between the rows (Figure 12).

Shear plates in green timber members can present serious splitting problems, even with single plates, as the circular plates are of solid material and do not cater for any adjustment under the lateral forces imposed during shrinkage of green timber. In such applications the recessed grooves in the timber, which accommodate the shear plates, need to be machined oversize to permit shrinkage free of restraint. The additional slip in the joints, due to clearance in the grooves, needs to be allowed for in design.

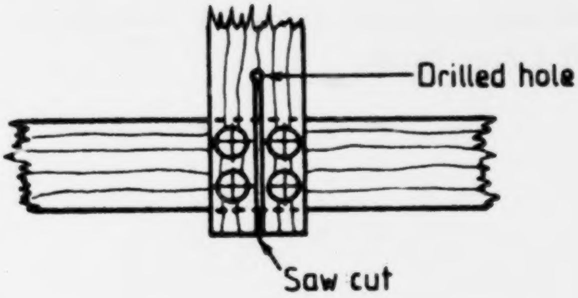


Figure 12   Use of a saw-cut to reduce end-splitting in green members subjected to lateral restraint

The variety of examples described should serve to indicate that the use of green timber need not present serious problems because of shrinkage. Entirely satisfactory structures of all types may be built of this material. It is only necessary to visualize what is going to happen to a structure or to a joint when green timber shrinks and, where necessary, make a simple provision for shrinkage.

BIBLIOGRAPHY

Kauman, W.G. (1964). Cell collapse in wood Holz als Roh-und Werkshoff 22(5) : pp.183-196, and 22(12) : pp.465-472.

Kingston, R.S.T., and Risdon, C.J.E. (1961). Shrinkage and Density of Australian and Other South West Pacific Woods. CSIRO Aust. Division of Forest Products Technological Paper No. 13.

Kloot, H. (1959). Shrinkage in Structural Timbers. CSIRO Forest Products Newsletter No. 256.

Pearson, R.G., Kloot, N.H., and Boyd, J.D. (1958). 'Timber Engineering Design Handbook'. CSIRO Australia. Melbourne Univerity Press.

Timber Promotion Committee and Timber Merchants' Association, Melbourne, Vic. (1974). Technical Timber Guide.

POLE STRUCTURES

G. B. Walford[1]

INTRODUCTION

The most common use of poles in developed countries is for the support of overhead transmission lines although in New Zealand this market has, over the past decade, been eclipsed by a demand for poles for house building. By using poles for the foundations as in pole platform type construction, or as part of the framework as well as in pole frame construction, previously unuseable land has been built on. This fact may be viewed with some amusement by residents of many tropical Pacific nations where for centuries houses have been built on pole foundations for reasons of safety from intruders, animals and earthquakes, and to improve ventilation. Substantial savings in building costs are obtained by using poles as columns with the butt end embedded in the ground. This application is frequently seen in warehouses. Poles are also used for bridge stringers, shelter fences,retaining walls, wharf piles and lookout tower structures. With a little ingenuity in fabricating joints, poles have been used for beams and rafters in conjunction with pole columns.

Poles have several inherent advantages over sawn timber: they are often the most readily available form of structural timber, particularly in remote areas; the costs and waste of sawing are eliminated; a pole is always stronger than the laminated beam that can be made from it; and for reasons to be discussed, higher allowable stresses may be assigned to poles than to sawn timber. Also, trees such as those from plantation thinnings or left as waste in clearfelling operations because they are too small to be utilised economically as sawlogs may be used quite effectively in pole structures, even temporary ones such as the scaffolding seen in many Asian countries.

Poles have two particular disadvantages: their shape and, for many species, poor durability in ground contact. The development of architectural and constructional techniques for handling the aesthetics and round shape of poles, and the development of a preservation industry that can supply treated pole timbers with a guaranteed life in excess of 80 years even in ground contact, has done much to overcome these difficulties. Probably the most outstanding pole frame building in the world is the indigenous performing arts centre in Nairobi, Kenya, which contains a pole frame dome spanning 120' (36.6 m).

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Useful information on the structural aspects of pole frame buildings is given in the AITC manual (1) and the report by Patterson and Kinney (2). Guidance on the design of simple pole frame buildings is available from Bournon and Keating (3), and NZTRADA. (4). Publications on more elaborate pole frame housing have been produced by Degenkolb (5), Norton (6), and Blakey (7). An excellent dossier describing the design and cost of various pole structures has been produced by Lattey (8).

DESIGN STRESSES

The structural characteristics of poles have been previously described to this workshop. The system of assigning stress grades to round timbers adopted in AS1720 is convenient and the comparison between these and the stress grades assigned to select grade sawn timber is as shown in Table 1.

TABLE 1. Stress grades for poles and round timbers

Strength group	Stress grade for green timber	
	Pole	Select grade sawn
S1	F34	F27
S2	F27	F17
S3	F22	F17
S4	F17	F14
S5	F14	F11
S6	F11	F8/F7
S7	F8	F7

The poles are assigned one stress grade higher than the highest grade of sawn timber because the occurrence of natural defects is compensated by the following:

(a) the tension strength of pole timber is increased by the fact that fibres flow smoothly around natural defects and are not terminated as sloping grain at sawn faces,

(b) many hardwood species have large tension growth stresses around their perimeters, and this assists in increasing the bending strength of the compression face of a pole in bending. (The detrimental effect of the growth stresses on the tension face is not important due to the large tension strength of pole timbers),

(c) in pine species there is usually an increase in wood density and hence strength properties from the pith to the bark. Thus the highly stressed wood in a pole in bending is likely to be above the specie's average in its properties.

The equivalence expressed in Table 1 is based on the assumption that poles or logs are from mature trees. If the tree is less than 25 years when felled, or the pole has less than 25 growth rings at the butt end, then an adjustment should be made for the effects of immaturity. This effect is shown in the data given in Table 2.

TABLE 2. Effect of age on the strength of green radiata pine poles (after Boyd (9))

Age of tree years	Modulus of rupture MPa
12	39.3
16	38.6
20	46.2
22	43.4
29	52.4
32	52.4

A factor predicting a similar effect of maturity on radiata pine poles was deduced from the variation of density with age and its relationship to strength by Walford (10) as follows:

No. of growth rings:	10	15	20	25
Factor relative to age 15:	0.88	1.00	1.09	1.15.

Another consideration for poles is the effect on strength of bark removal. If the bark is removed manually, e.g. using a spade, machete or drawknife, then it is likely that little damage will be done to the swellings in the trunk around knots. If the bark is removed by machine, and the resulting pole has a smooth tapering profile then some strength loss will be incurred because the natural reinforcement around the knots will have been removed and sloping grain exposed. The degree of damage depends on the amount of swelling initially present. Table 3 gives the losses in modulus of rupture and modulus of elasticity in Corsican pine poles which had an average nodal swelling of 20 mm at each knot whorl and a mean diameter between nodes of 220 mm.

TABLE 3. Percentage loss in strength and stiffness of poles due to wood removal from nodal swellings (after Walford (11))

Depth of wood removal (mm)	Percentage loss in	
	MOR	MOE
5.75	14	6
15.5	29	15

Since any soil can pick up moisture, the design stresses chosen for embedded poles at the ground line must always be taken to be those of green timber whereas for dry poles, e.g. in a roof structure, 10 to 20% higher stresses are reasonable according to Boyd (9).

DESIGN OF GROUND EMBEDMENT

Poles embedded in the ground may be designed as cantilevers resisting lateral loads. The required depth of embedment may be calculated from Appendix K of AS1720 if the poles are relatively short. For larger poles the procedure developed by Rutlege is useful and is given in references (1) and (2). This procedure is based on the criterion of a limiting movement of 1/2" (12 mm) at ground level on application of design load. Figure 1 is reproduced from reference (2) and gives the required depth of embedment for a pole without rigid restraint at the groundline, e.g. by a concrete floor. It is based on the formula:

$$D = A\,(1 + (1 + 2.18H/A)^{0.5}) \qquad (1)$$

where A = 2.34 P/SB

H = height above groundline at which force P acts

P = horizontal force acting on the pole

S = soil pressure at depth 0.34 D below ground level

B = diameter of pole, or of encasing reinforced concrete, or of hard backfill.

If rigid restraint is provided at ground level, the depth of embedment is given by:

$$D = (4.25\ PH/SB)^{0.5} \qquad (2)$$

where S is the allowable lateral soil pressure at the depth D below ground level.

The allowable soil pressure, S, at any specified depth, h, below the soil surface is the maximum soil passive pressure that can develop. For most soils that are suitable for foundations a value of S = 12 kPa (250 psf) is conservative. For sand the passive pressure may be taken from Rankines formula:

$$S = \frac{1 + \sin\theta}{1 - \sin\theta}\ 19\,h\ \text{kPa} \qquad (3)$$

where h is in metres

and θ is the angle of friction of the sand.

For more general cases S may be taken to be the lesser of S_1 and S_2 given in Table 4.

TABLE 4. Allowable lateral soil pressure at depth h (from ref. 2)

Class of material	S_1	S_2 (kPa)
Good - Compact well graded sand and gravel, hard clay, well graded fine and coarse sand (all drained so water will not stand)	60 h	380
Average - Compact fine sand, medium clay, compact sandy loam, loose coarse sand and gravel (all drained so water will not stand)	30 h	120
Poor - Soft clay, clay loam, poorly compacted sand, clays containing large amounts of silt (water stands during wet season)	15 h	70

POLE CONNECTIONS

Connections are usually the most difficult aspect of the design of pole structures. Some typical details are shown in Figure 2 while Figures 3 and 4 show less conventional but nevertheless effective connections using steel strapping or threaded steel rods. In the case of the strapping the same result may be obtained using wire or some form of lashing.

Poles treated with copper chrome arsenate preservatives have an additional problem with corrosion of fasteners if moisture is present. Three methods have been used to overcome this:

(a) Use stainless steel strapping or bolts etc.

(b) Use galvanised steel bolts protected by plastic tubing.

(c) Use galvanised steel bolts and hardware liberally coated in grease.

EXAMPLES

Figure 5 gives schematic diagrams of various structures described in detail in references (3), (5) and (8), as follows:

(a) Shows a simple monopitch frame which relies on the embedment of the poles for its resistance to lateral loads.

(b) The tied eaves portal design reduces the reliance on cantilever action of the pole columns. Steel rod or wire makes a satisfactory tie which needs provision for tightening e.g. a turnbuckle in the rod or Spanish windlass in the wire.

(c) The two bay unbraced or tied portal could be extended to as many bays as desired. This design can be used to produce a round structure with a single central pole.

(d) Trusses on poles is a common design for haybarns in New Zealand.

(e) Pole platform construction quickly provides a level support for conventional light frame construction.

(f) Pole frame construction results when the poles are carried through to support the roof and/or the building is on several levels down the slope.

(g) A simple scissor truss can be made using slender poles and bolts.

REFERENCES

1. ANON, 1966: Timber Construction Manual, prepared by American Institute of Timber Construction, John Wiley and Sons Inc., New York, pp. 4.83-4.93.

2. PATTERSON, D. and KINNEY, E.E. 1969: Pole building design, American Wood Preservers Institute.

3. BOURNON, R.N. and KEATING, W.G. 1971: Designs for pole frame buildings, Forest Products Technical Notes No. 7, CSIRO.

4. ANON, 1972: Pole frame buildings, NZTRADA Timber and Wood Products Manual Section 2b-1.

5. DEGENKOLB, H. et al. 1975: FHA pole house construction, American Wood Preservers Institute.

6. NORTON, P. 1976: The New Zealand pole house, Hicksons Timber Impregnation Co (NZ) Ltd.

7. BLAKEY, J. 1973: Pole type construction, NZTRADA Timber and Wood Products Manual Section 2f-1.

8. LATTEY, P. 1974: Pole buildings in Papua New Guinea, Forest Products Centre, PO Box 1358, Boroko, PNG.

9. BOYD, J.D. 1964: The strength of Australian pole timbers - IV Radiata pine poles, Forest Products Technicological Papers No. 32, CSIRO.

10. WALFORD, G.B. 1975: Design criteria and specification of poles, presented to seminar "Roundwood products of the future". NZTRADA.

11. WALFORD, G.B. 1981: The effect of mechanical debarking on the strength of Corsican pine poles, NZFS, Forest Research Institute, FP/TE report No. 38 (unpublished).

Chart for Embedment of Posts

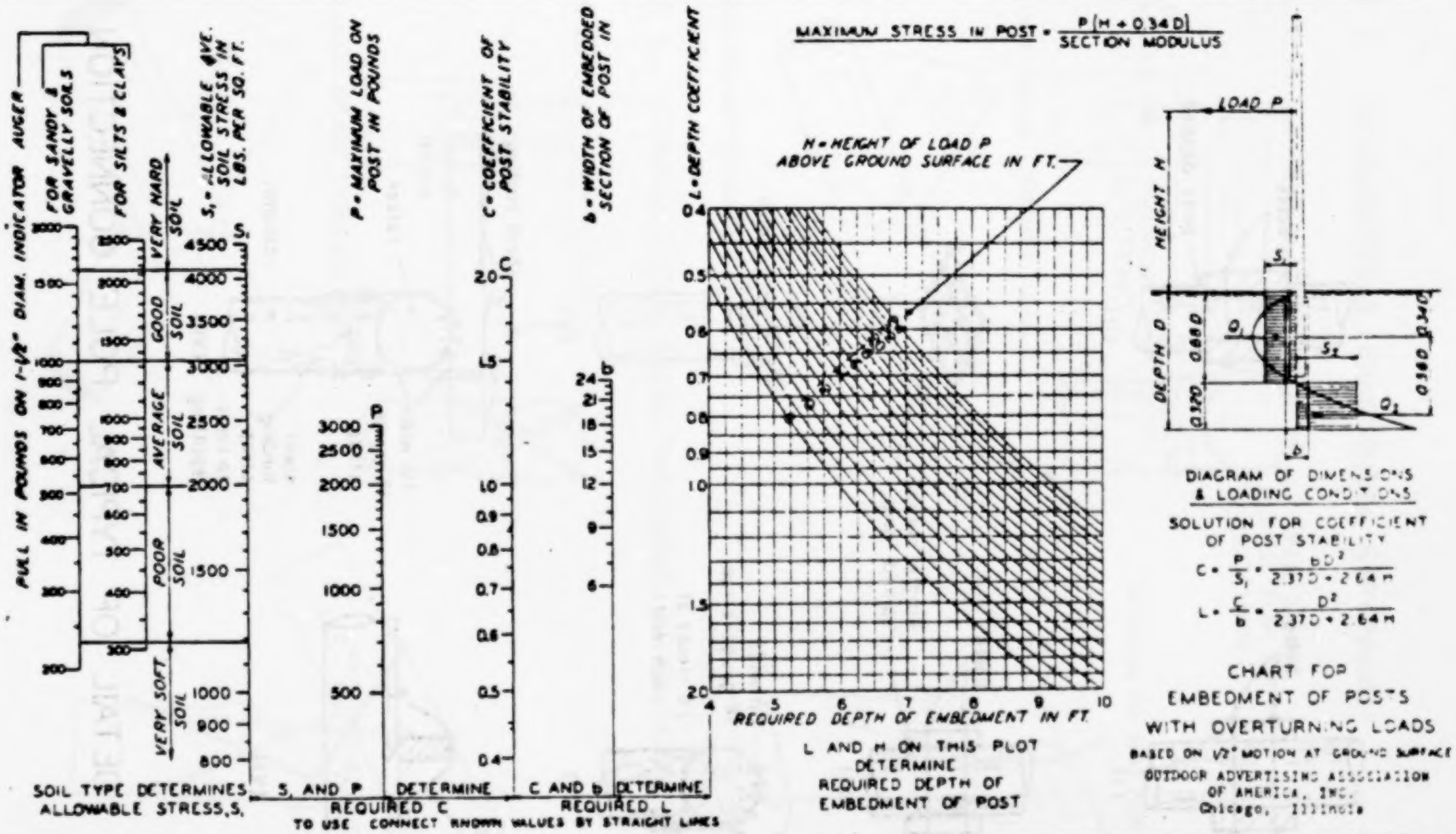


Figure 1

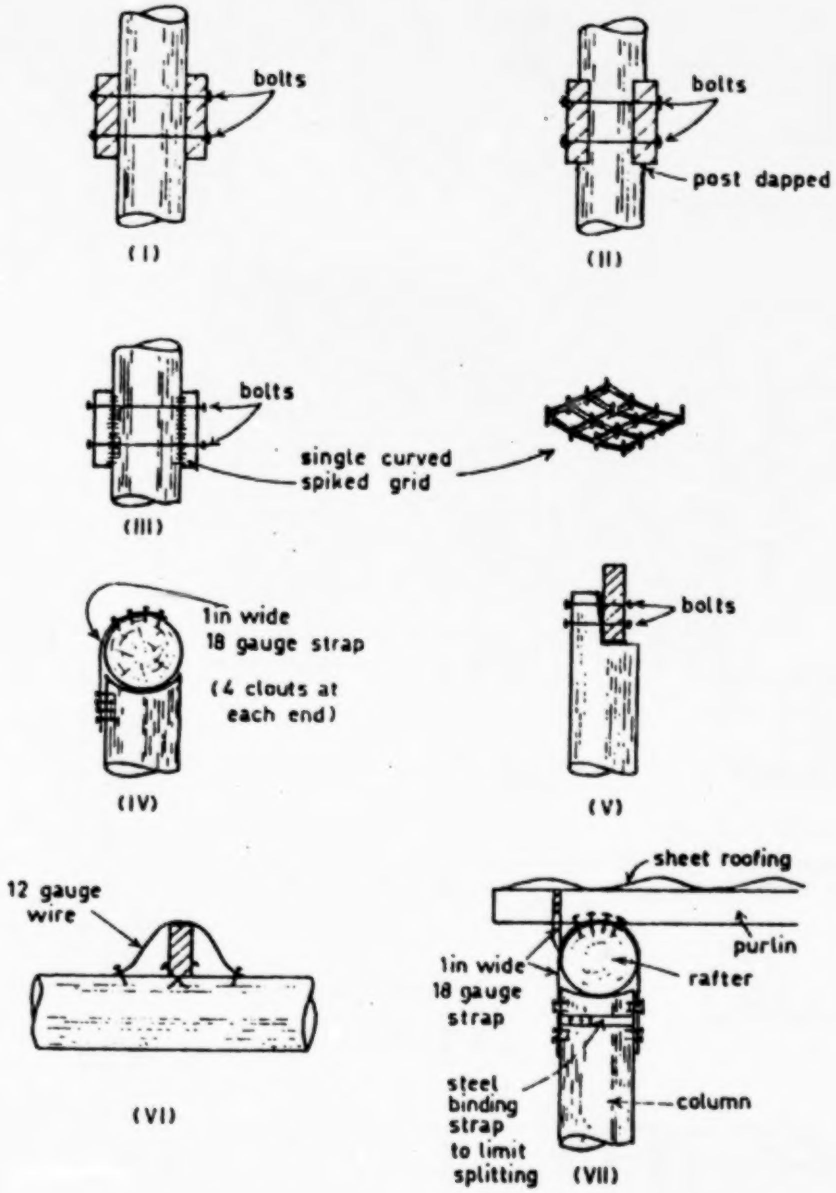
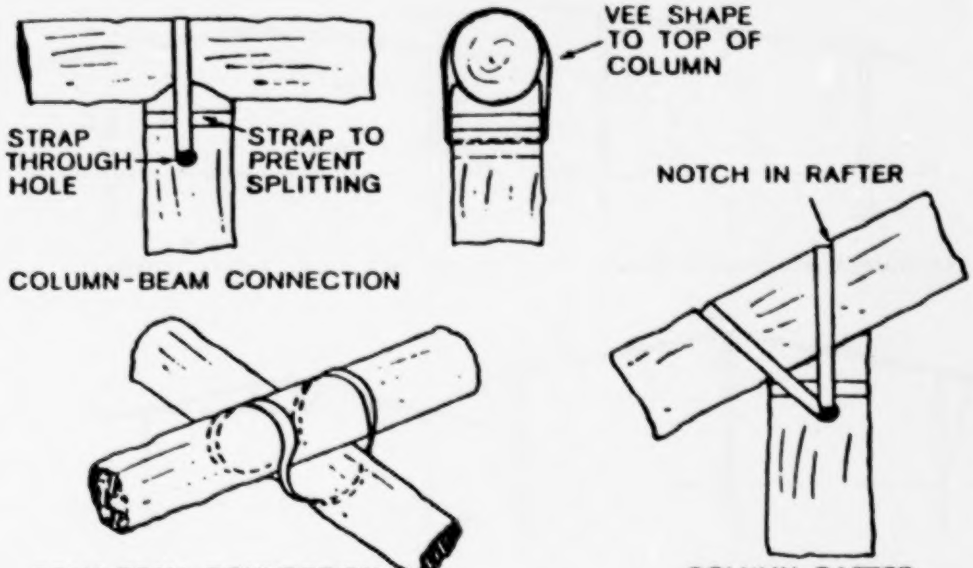

bolts
(I)
bolts
post dapped
(II)
bolts
single curved
spiked grid
(III)
1in wide
18 gauge strap
(4 clouts at
each end)
(IV)
bolts
(V)
12 gauge
wire
(VI)
sheet roofing
purlin
1in wide
18 gauge
strap
rafter
steel
binding
strap
to limit
splitting
column
(VII)

FIG.2 DETAIL OF TYPICAL POLE CONNECTION

FIGURE 3. CONNECTIONS USING STEEL STRAPPING

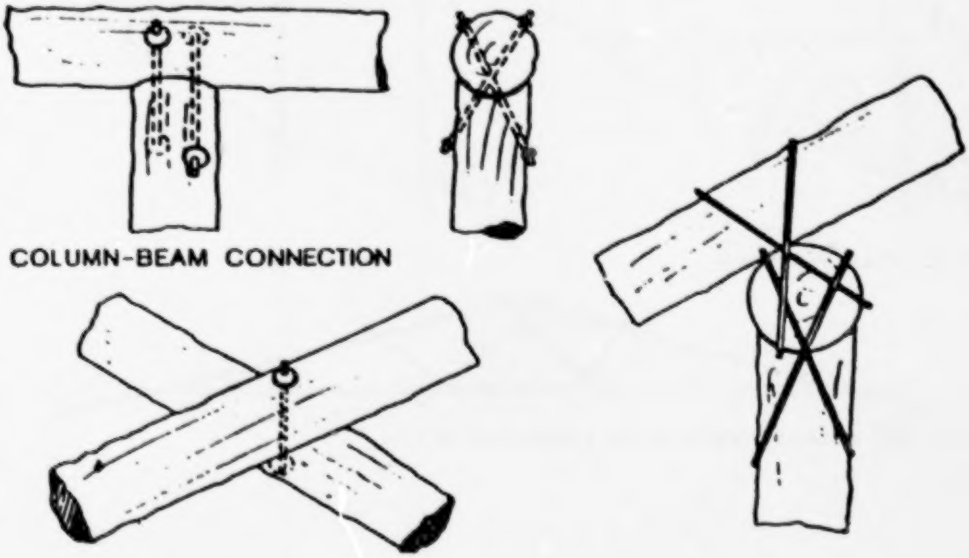

STRAP THROUGH HOLE
STRAP TO PREVENT SPLITTING
VEE SHAPE TO TOP OF COLUMN
NOTCH IN RAFTER

COLUMN-BEAM CONNECTION

BEAM-BEAM CONNECTION

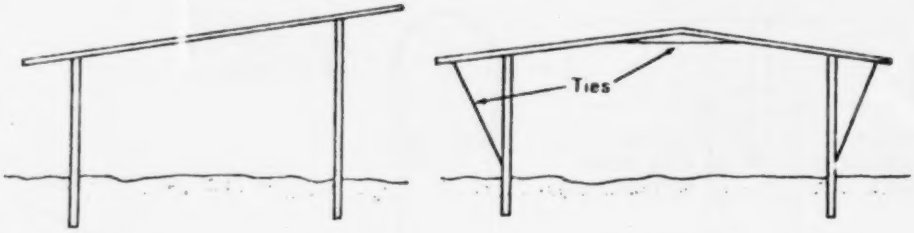
COLUMN-RAFTER CONNECTION

FIGURE 4. CONNECTIONS USING THREADED STEEL ROD



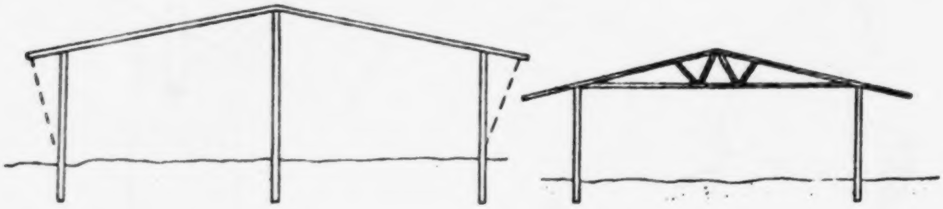
COLUMN-BEAM CONNECTION

FIG. 5 EXAMPLES OF POLE STRUCTURES



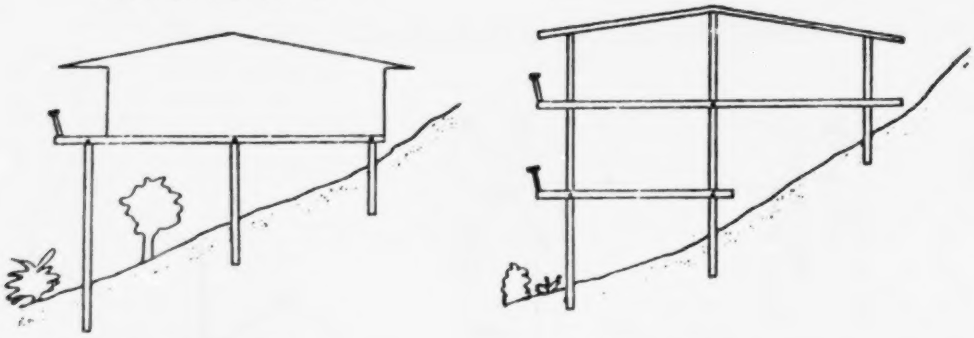
(A) SIMPLE MONOPITCH FRAME

(B) TIED EAVES PORTAL FRAME



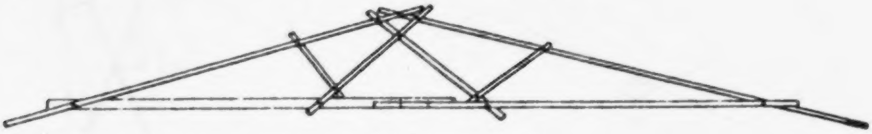
(C) TWO BAY UNBRACED OR TIED PORTAL FRAME

(D) POLES SUPPORTING TRUSSES



(E) POLE PLATFORM

(F) POLE FRAME



(G) W TRUSS MADE FROM POLES AND BOLTS

TIMBER FRAMING FOR HOUSING

Bernie T. Hawkins[1]

1. INTRODUCTION

In Australia about 80% of housing is constructed with a basic frame usually of timber, lined internally with some sort of plasterboard and clad externally most often with a veneer of bricks. About 15% are clad externally with timber boards or cement sheet boards.

House framing represents by far the largest use of structural timber in Australia. Australian Standard 1684 Timber Framing Code and its predecessor have been used for many years to ensure that safe, satisfactory houses and other buildings are erected in the most economical manner possible.

In this paper the code is looked at in detail and other factors involved in the building and performance of house frames are considered.

The terminology and method of construction used is illustrated in Fig.1.

2. HISTORY

In 1941 the first edition of the CSIRO Division of Forest Products Pamphlet 112 appeared. Entitled 'Building Frames - Timber and Sizes', this document was very simple with very few variations for spacing or loadings, etc. Nevertheless, as at that stage there was little variation in the type of house being built, it was in great demand from architects, builders, specifiers, etc. A revised and expanded edition was brought out in 1952 but by the late sixties it was obvious with the changes in strength grouping and stress grades, together with the variety of houses being built, that a more comprehensive document was needed. This document was CA 38, the Light Timber Framing Code, first published by the Standards Association of Australia in 1971. Since that time the code has been metricated and revised on four occasions, until today it is AS 1684-1979, Timber Framing Code. As an Australian Standard, this code was written under the direction of a committee comprising mainly builders, timber suppliers, wood technologists and representatives of timber and building associations and specifiers.

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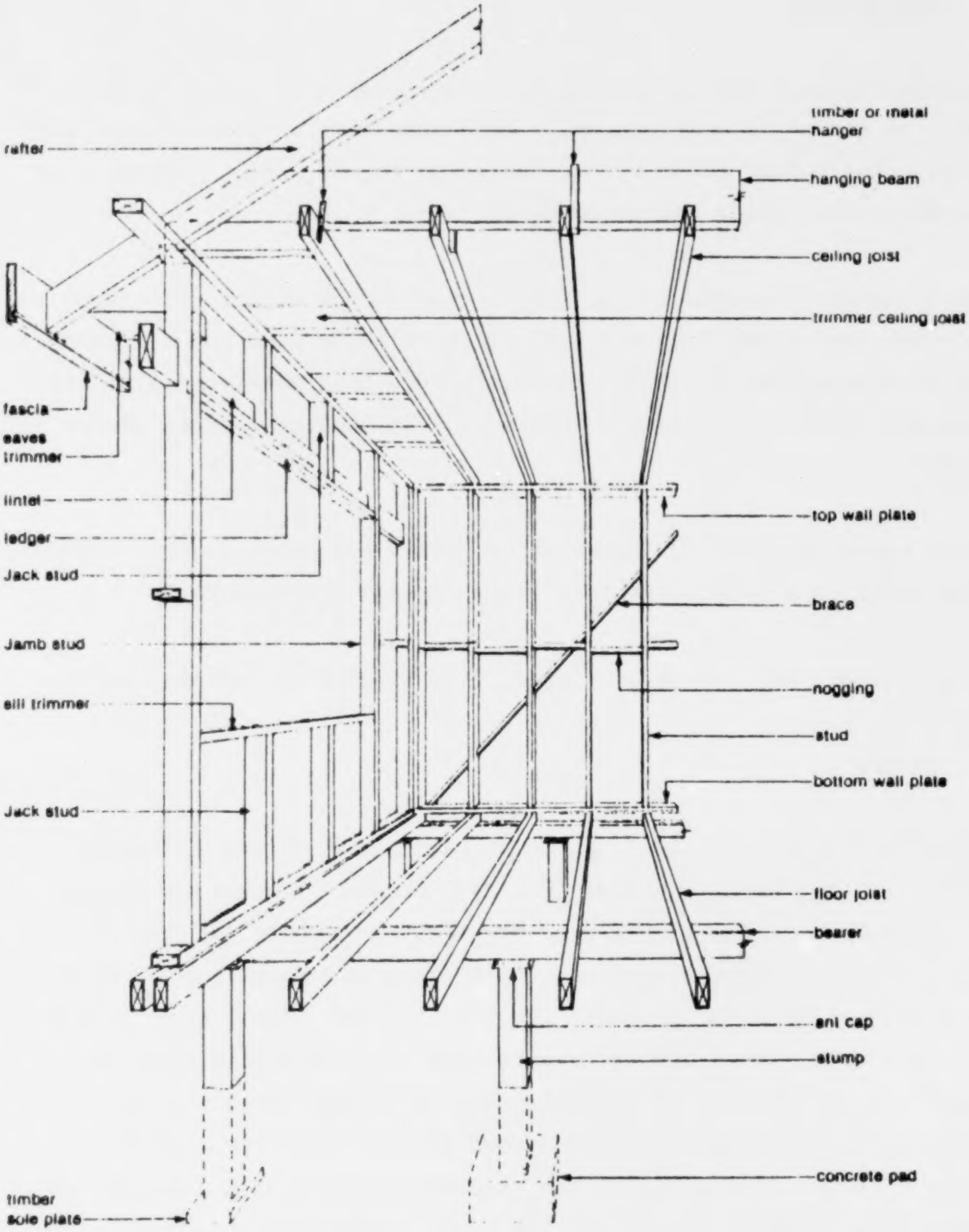
rafter
timber or metal
hanger
hanging beam
ceiling joist
trimmer ceiling joist
fascia
eaves
trimmer
lintel
ledger
Jack stud
top wall plate
brace
Jamb stud
nogging
sill trimmer
stud
bottom wall plate
Jack stud
floor joist
bearer
ant cap
stump
concrete pad
timber
sole plate

Figure 1 Nomenclature of framing members

3. CONTENT

The code itself is a 56 page document divided into 6 sections as follows

Section 1 - Scope and General
Section 2 - Substructure
Section 3 - Timber Floor Framing and Flooring
Section 4 - Timber Wall Framing
Section 5 - Timber Roof Framing
Section 6 - Nailing and Anchorage.

In addition, the main document contains four appendices dealing with the properties and grades of Structural Timber, Site Preparation, Storage and Handling of Timber and Precaution against wind effects.

In Section 1, as might be expected, terms are defined and general rules given for such things as interpolation between values in the Tables and other information on the use of Tables. In addition, advice is given on storage and handling of timber.

Section 2 gives details of the site preparation and foundation necessary for frame construction. It also gives information on such things as subfloor ventilation and termite protection.

Sections 4, 5 and 6 deal with the selection and placement of the various members throughout the frame. Details are given on such things as bracing requirements (both temporary and permanent), drilling and notching of members, etc., and other details of workmanship. This is a very important part of the code and will be discussed further.

Section 6 gives the number and diameter of nails used in various parts of the frame. It also discusses alternative methods of fixing in some situations.

However, several other parts of the code are issued as supplements. Each of these supplements contains the details of spans, etc., capable of being supported by various sizes of timber under various spacings, etc., for timber of one stress grade and one moisture class. The various members covered in each supplement are shown in Table 1.

TABLE 1
LIST OF TABLES IN TIMBER FRAMING CODE SUPPLEMENTS

Table 1S(A)	Bearers supporting single-storey loadbearing walls - Maximum spacing of bearers parallel to wall 1.8 m
Table 1S(B)	Bearers supporting single-storey loadbearing walls - Maximum spacing of bearers parallel to wall 3.6 m
Table 2S	Bearers supporting floor joists only
Table 3S	Floor joists
Table 4S(A)	Studs for single-storey loadbearing walls - Studs spaced up to 450 mm apart
Table 4S(B)	Studs for single-storey loadbearing walls - Studs spaced up to 600 mm apart
Table 5S	Studs supporting concentration of loding
Table 6S	Studs at sides of openings - Single-storey constructions or upper storey of two-storey constructions
Table 7S	Top wall plates - Single or upper storey
Table 8S	Bottom wall plates - Single or upper storey
Table 9S	Lintels - Single or upper storey
Table 10S	Ceiling joists supporting ceiling only
Table 11S	hanging beams for ceiling joists
Table 12S	Hanging beams for ceiling joists
Table 13S(A)	Rafters or roofing purlins - Supported at two points only
Table 13S(B)	Rafters or roofing purlins - Continuous over two or more spans
Table 14S	Roof beams (principals) for non-trafficable roofs
Table 15S	Strutting beams for roof members
Table 16S	Underpurlins
Table 17S	Roofing battens
Table 18S	Verandah posts
Table 19S(A)	Bearers supporting two-storey loadbearing walls - Maximum spacing of bearers parallel to wall 1.8 m
Table 19S(B)	Bearers supporting two-storey loadbearing walls - Maximum spacing of bearers parallel to wall 3.6 m
Table 20S	Bearers cantilevered to support balconies
Table 21S	Joists for upper floors and permissible cantilever to support balconies

TABLE 1 (continued)

Table 22S(A)	Studs for upper storey loadbearing walls - Studs spaced up to 450 mm apart
Table 22S(B)	Studs for upper storey loadbearing walls - Studs spaced up to 600 mm apart
Table 23S(A)	Studs for lower storey loadbearing walls - Studs spaced up to 450 mm apart
Table 23S(B)	Studs for lower storey loadbearing walls - Studs spaced up to 600 mm apart
Table 24S	Studs at sides of openings in lower storey loadbearing walls
Table 25S	Top wall plates for lower storey loadbearing walls
Table 26S	Bottom wall plates for lower storey loadbearing walls
Table 27S	Lintels in lower storey loadbearing walls

There are 22 supplements at present, ranging from unseasoned timber of F4 grade to seasoned hardwood of F27 grade. The titles of the various supplements are shown in Table 2.

TABLE 2
LIST OF SUPPLEMENTS OF LIGHT TIMBER FRAMING SPAN TABLES

Supp. No. 1	Unseasoned Timber - Stress Grade F4
Supp. No. 2	Unseasoned Timber - Stress Grade F5
Supp. No. 3	Unseasoned Timber - Stress Grade F7
Supp. No. 4	Unseasoned Timber - Stress Grade F8
Supp. No. 5	Unseasoned Timber - Stress Grade F11
Supp. No. 6	Unseasoned Timber - Stress Grade F14
Supp. No. 7	Unseasoned Timber - Stress Grade F17
Supp. No. 8	Unseasoned Timber - Stress Grade F22
Supp. No. 9	Seasoned Softwood - Stress Grade F5
Supp. No. 10	Seasoned Softwood - Stress Grade F7
Supp. No. 11	Seasoned Softwood - Stress Grade F8
Supp. No. 12	Seasoned Softwood - Stress Grade F11
Supp. No. 13	Seasoned Hardwood - Stress Grade F11
Supp. No. 14	Seasoned Hardwood - Stress Grade F14
Supp. No. 15	Seasoned Hardwood - Stress Grade F17
Supp. No. 16	Seasoned Hardwood - Stress Grade F27
Supp. No. 17	Unseasoned Timber (Alternative Sizes) - Stress Grade F4
Supp. No. 18	Unseasoned Timber (Alternative Sizes) - Stress Grade F5
Supp. No. 19	Unseasoned Timber (Alternative Sizes) - Stress Grade F8
Supp. No. 20	Unseasoned Timber (Alternative Sizes) - Stress Grade F11
Supp. No. 21	Seasoned Softwood - Stress Grade F4
Supp. No. 22	Seasoned Softwood - Stress Grade F14

4. USE OF CODE

To establish member sizes and/or spans for the various parts of a framed structure, it is first necessary to refer to the appropriate section of the main code. For example, if we wish to find a suitable ceiling joist to span 2.4 m in dry radiata pine of F5 stress grade, we would refer to section 5.2.2. Here, amongst information on spacing, direction, splicing and method of support, we are told that Table 10S is the table in the appropriate supplement which will give us the size of material necessary to span 2.4 m. However, on viewing Table 10S in Supplement No. 9 which is for F5 sesoned softwood, it can be seen that we must first decide the spacing of the joists. If the joists are to be supported at two points only, then the table shows that if the spacing is 600 mm it is necessary to use a 120 x 35 joist. This table also shows us, however, that if we wish to put a hanging beam halfway across the span, then we could use 70 x 45 joists as these are capable of spanning 1.4 m and in this case we would only be looking to span 1/2 of 2.4 m.

It is possible of course to start with a given timber size and use the table to find the maximum span it can carry.

Some members, particularly wall members, are not as easily defined as their size, etc., are determined mainly by the amount of roof load they have to carry. Consequently it has become necessary to introduce a variable which will define the amount of roof load carried by a wall member. This variable is called Effective Roof Length or EL and is defined by Figure 7 in the code. It must be stressed that although Figure 7 shows roofing members, it has nothing to do with the actual roof itself but is merely a means of defining how much load goes onto the wall members from the roof. An estimate of the load on an external wall due to the roof weight is obtained by (unit width of wall) x (half of EL plus length of eaves overhang) x (mass of roof per unit area).

Overall, in using the code it can be seen that the tables offer a wide range of options for the designer and/or builder in sizes, spans and stress grades, and that the main body of the code gives a standard of workmanship which is expected if the tables are to be used efficiently.

5. TECHNICAL BASIS OF THE CODE

A reasonably detailed description of the technical matters involved in producing the tables in the code is contained in the publication 'Low-Rise Domestic and Similar Framed Structures - Part 1', which is published by CSIRO Division of Building Research. Here I will give a brief summary of some of the major factors involved.

(a) Dead Loads. Dead loads are calculated from the masses of various building materials given in AS 1170 - Part 1, SAA Loading Code. However, some modifications are made to the self-weight factor to allow for the higher stress grades usually coming from more dense species.

(b) Live Loads. Live loads are those specified by AS 1170 for various parts of the structure.

(c) Wind Loads. The wind loading adopted for calculations in AS 1684 is that due to an effective wind velocity of 33 m/sec. The effective wind velocity is obtained by establishing the regional basic wind velocity for a given return period and multiplying it by a factor known as the terrain category factor dependent on the height of the structure and the nature of the surroundings. For Brisbane in the built-up metropolitan area, the relevant figures are 50 m/sec for a regional basic wind velocity with a 50 year return period, and a multiplying factor of 0.65 for a terrain Category 3, up to 5 m height of buiding. This represents the upper limit of 33 m/sec used in the code.

(d) Design Stresses. A further paper in this workshop on Timber Engineering explains how the basic working stresses are derived for each stress grade from the known strength properties of the timber. However, before these allowable stresses are used, they can be modified to suit the particular situation in which the timber is to be used. The modification factors are all discussed in the Timber Engineering Code AS 1720, but I will briefly mention some of the more important ones occurring in the Timber Framing Code.

(i) Duration of load - many loads involved in the code are of short duration, e.g. a man carrying a heavy weight, or a local dense crowd. In these cases, the working stress may be 56% higher than for permanent loads. In the case of the shortest load, i.e. wind load, the working stress is allowed to be doubled.

In the calculation of member deflections under dead loads, the short-term deformation requires to be multiplied by a factor of 3 in the case of initially green timber, or by 2 if the timber is seasoned before use.

(ii) Moisture content - most framing timbers used in the green or partly dry condition are of a size that drying to equilibrium moisture content occurs within a year. Tests have shown that hardwood species in Australia generally increase in load carrying capacity by about 25% on drying, despite the reduction in size due to shrinkage. To take this into account, the working stresses have been increased by 15% for members up to 38 mm thick, 10% for members 50 mm thick and 5% for 75 mm thick members. Above this thickness it is considered drying would be too slow for any allowance to be made to working stresses.

(iii) Temperature - variations in ambient temperatures throughout most of Australia is usually accompanied by a compensating variation in the moisture content of the timber. For this reason, although timber gets weaker with increasing temperature, it is only in the northern coastal regions where high temperatures coincide with high humidities that it is necessary to take some action. In this case, the basic working stresses are reduced by 10%.

(iv) Load sharing - where a group of members act together as a system, two factors operate to assist the system. Firstly, if one member of the group is weaker than the others and reaches its ultimate load-carrying capacity, the system may continue to sustain an increasing load because the adjacent members are stiffer and stronger. Allowance is made for this effect by allowing an increase of 15% in working stresses for members spaced at 450 mm diminishing to no increase for members spaced at 1200 mm or more apart. Secondly, where members of a system cannot deflect independently of other members, such as in a 2 or 3 layer grid system, localised loads are distributed laterally to more members than those directly under the load. The effective concentrated load which must be considered in these cases is given in Clause 3.2.7 of AS 1720 Timber Engineering Code.

(e) Minimum Timber Dimensions. The minimum dimensions of timber allowed when the spans specified in the code are used are shown on each Table in the supplements. In general, green timber can be 3 or 4 mm under the size nominated, but dry timber cannot be below the nominated size.

(f) Design Criteria. Virtually all the members of a framed structure, with the obvious exception of members such as stumps and struts, act as beams, wall studs being treated as beam-columns. The maximum allowable span is determined as the minimum value of the following:

(i) Maximum span of adequate bending strength under dead loads alone, including the self-weight of the member, and under the combined dead and live loading.

(ii) Maximum span of adequate shear strength under the same loading conditions.

(iii) Maximum span with acceptable long-term deflection under live loads.

For roof members:

(iv) Maximum span of adequate bending strength under combined dead and wind loads.

For the case of wall members:

(v) Also maximum span of adequate bending strength under the combined vertically applied dead and wind loads, together with the transversely applied wind load.

The tables in Part 1 of Low-Rise Domestic and Similar Framed Structures give full detail of all these criteria. However, Table 3 gives a brief summary of the deflection criteria for the major members in a house frame.

TABLE 3
DEFLECTION CRITERIA

Member	Maximum permissible deflection	
	Dead load only	Live load only
Members subjected to foot traffic	span/300, max. 12 mm	span/360, max. 9 mm
Rafters and purlins	span/300, max. 20 mm	span/300, max. 12 mm
Ceiling joists and hangers	span/300, max. 12 mm	span/270, max. 15 mm
Strutting beams supporting rafters and ceiling joists	span/300, max. 12 mm	span/300, max. 12 mm
Wall plates	span/240, max. 6 mm	span/240, max. 6 mm
Lintels	span/300, max. 9 mm	span/240, max. 9 mm
Studs	span/300, max. 9 mm	span/360, max. 9 mm (live roof load) span/240, max. 12 mm (wind load)

6. FUTURE OF THE CODE

The code is being continuously reviewed by a committee of the Standards Association. This committee has at present commissioned a sub-committee to look at a major revision for two to three years hence. It is expected that some extra material which is now available in Part V of the CSIRO series on Low-Rise Domestic and Similar Framed Structures will be incorporated.

7. WORKMANSHIP

The Timber Framing Code assumes that workmanship is of such a standard that all members are capable of resisting the forces assumed in the engineering calculations used in developing the span tables, etc. In general, the standard of work of carpenters involved in house framing is reasonable, however there are occasional examples of difficulties, particularly where later trades modify structural members. In general, it is up to the building inspector to detect gross departures from good building practice or good quality workmanship. The following examples are taken from a series of articles by Mr N H Kloot which appeared in the Forest Products Newsletter No. 400, 1975, which used to be published by CSIRO Division of Building Research. It should be pointed out that, in the cases shown, the building inspector had not yet inspected the building and would demand that the faults be corrected when detected by him.

7.1 The Barap Tie

The so-called Barap tie is now commonly employed as an effective means of increasing the strength and stiffness of a timber member such as a hip rafter when used over a rather large span. This tie is basically a steel rod fastened to each end of the timber member and propped away from it with a strut (Figure 2). The principle on which the Barap tie operates is by no means new and is well known to engineers as the king or queen post truss. When the truss is under load, the tie is stressed in tension and the whole structural unit becomes equivalent to a beam much deeper than the actual timber member used.

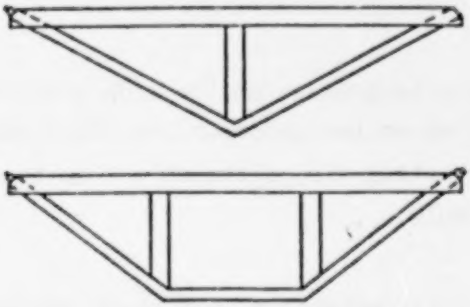

Figure 2 Use of the Barap tie to stiffen and strengthen a hip rafter. Trussed beam with king post (top) and with queen post (below)

Figure 3 illustrates a Barap tie fitted to a hip rafter, the strut being comprised of two small timber offcuts bearing on two underpurlins which, in turn, are nailed to the hip rafter. For practical purposes, the tie, as applied in this case, is absolutely useless. Between the steel rod and the hip rafter there is approximately 200 mm of side grain green timber. Even allowing a conservative 5% for shrinkage, the intended strut will shrink 10 mm. This will have the effect of completely unloading the steel tie so that the hip rafter itself will have to carry the full load. Obviously it is too small to do this for otherwise a Barap tie would not have been fitted in the first place, and so the rafter will eventually sag well beyhond the limits allowed for this type of member.

Figure 3 Green timber used on its edge as a prop for a Barap tie means that the tie becomes useless as the timber shrinks

Figure 4 shows another such installation, only in this instance a steel prop has been used.

Unfortunately, this prop is bearing on the side grain of a packing piece which, inturn, is bearing on two underpurlins. Here again, shrinkage will tend to unload the steel tie, although not quite to the same extent as in the previous example.

For a Barap tie to be fully effective, the strut should be of steel as in Figure 4 or a piece of timber loaded on end grain. Moreover, the top end of the strut should bear directly on to the timber member to which the tie has been fitted.

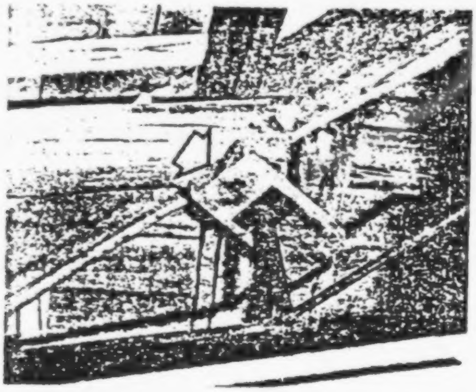

Figure 4 The steel prop goes part of the way towards making the Barap tie effective. Shrinkage of the packing piece, however, will tend to reduce the tie's effectiveness.

7.2 Notching for Braces

A stud, particularly in the outer wall, has to carry its share of the roof weight as well as resist horizontal wind forces even when the structure is clad with brick veneer. yet in spite of its importance, the stud is probably the most abused of all of the structural members.

Notches decrease the strength of studs and to some degree their stiffness, the decrease being greatest when the notch is near the centre of the height of the stud. In preparing the tables for notched studs in the Timber Framing Code, an allownce was made for loss of strength in a stud when the notch was no more than 5 mm deeper than the specified thickness of the brace.

Figure 5 shows a notch cut into a pine stud to a depth nearly twice the thickness of the brace. Furthermore, it can be seen that the stud has actually broken at a knot cluster immediately at the back of the notch. As a load-carrying member, this stud will be totally ineffective. The performance of the stud shown in Figure 6 would be well below that for which it has been designed after the plumber has provided himself with a notch immediately behind that carrying the timber brace.

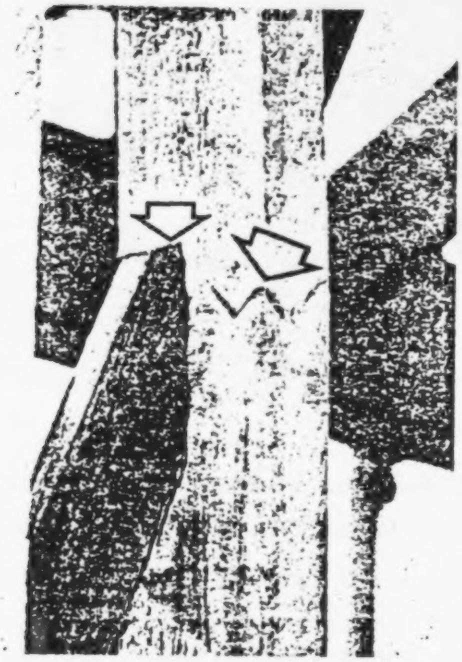

Figure 5 General notch in a stud. Note also failure of the stud at the knot cluster.

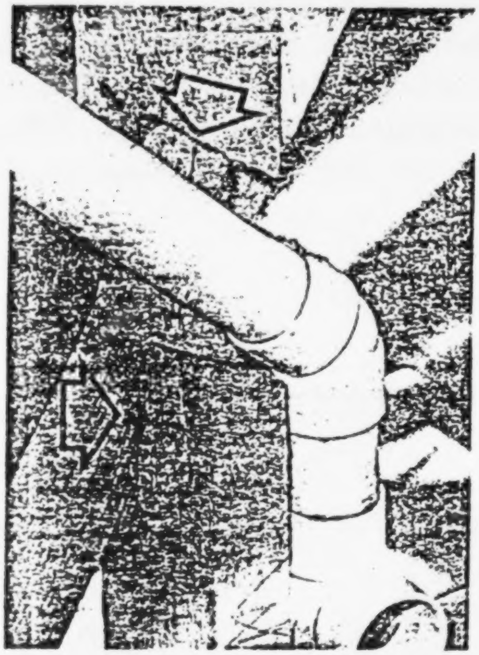

Figure 6 This might suit the plumber, but the stud's effectiveness as a roof support is almost negligible.

In Figure 7 the notch is longer than necessary and, although not clearly defined in the photograph, the initial sawcuts made to allow the timber to be removed for the notch are substantially deeper than required. This latter example of bad workmanship is well illustrated in Figure 8.

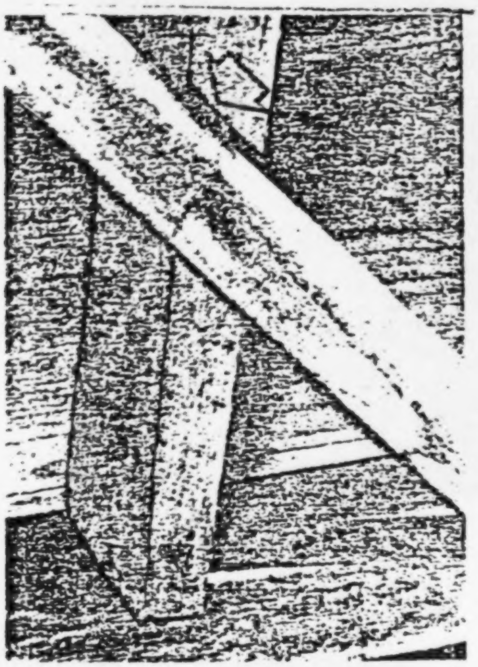

Figure 7 This stud's strength and stiffness has been seriously reduced by excessive and unnecessary notching.

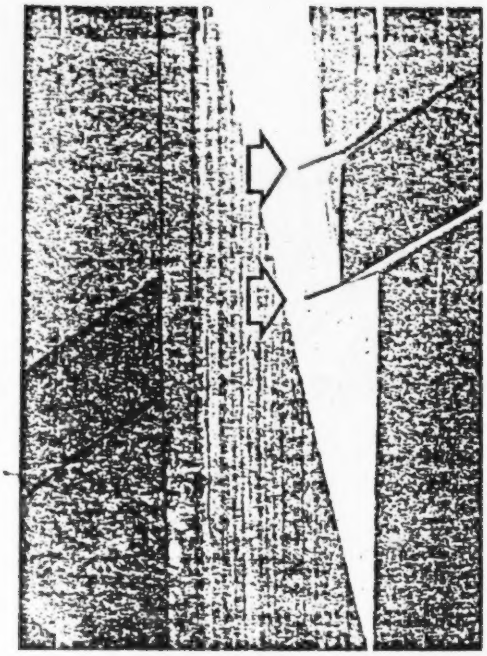

Figure 8 Another example of bad notching.

If the overcutting or overnotching occurred once or twice by accident in a whole house frame, the overall effect would not be serious. However, it usually happens that if one stud is overcut or overnotched, practically all the studs are similarly abused. This is a clear case of bad workmanship.

Even with frames constructed in accordance with Pamphlet No. 112, bad workmanship is critical. While this pamphlet recommends larger sizes for some members than those allowed in AS 1684, this is no safeguard against poor workmanship. In fact, the larger sizes may give the ill-informed building a misplaced sense of security when he comes to cutting notches, drilling holes, etc. in studs and other members. It is of interest to note that whereas P112 has no provisions to control quality of workmanship, AS 1684 has such provisions, including allowble depth of notches and sizes of drilled holes.

7.3 Overcutting

The advent of the portable electric saw has certainly made the job much easier for the on-site framer. At the same time, the overcutting of notches in studs, props and other members appears far more prevalent than it was. The tradesman with a hand saw was unlikely to overcut, except in error, because this took more time and extra physical effort.

Figure 9 shows the notch in a ridge prop overcut badly both horizontally and verticallly. With the subsequent drying of the prop, splitting from the end of the vertical cut is likely, in which case the prop will probably become largely ineffective.

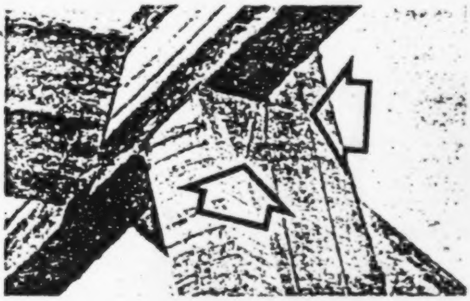

Figure 9 Excessive overcutting of notch in a ridge prop.

Figure 10 adds to the examples given in previously of gross overnotching of studs.

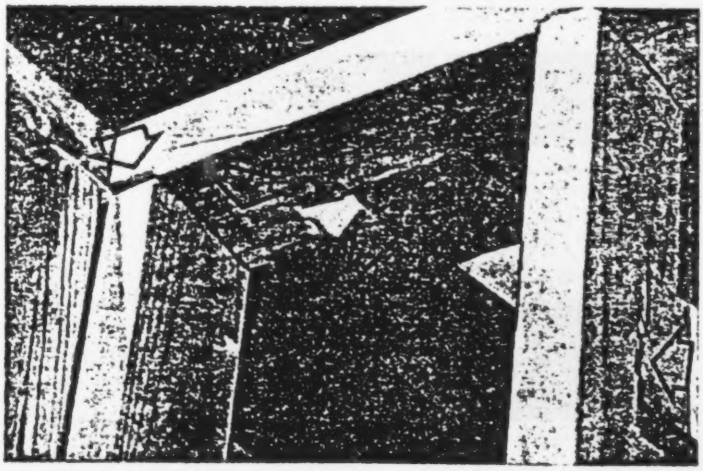

Figure 10 Another example of overnotching of stud and top plate

7.4 Making Do

Very frequently at a building site a relatively large pile of timber off-cuts accumulates and it is only reasonable that as much as possible of this off-cut material is put to sound practical use. However, improper use of this material on the basis of 'making do' or 'near enough is good enough', as illustrated in Figure 11, is definitely unsound building practice: Figure 12 shows an even worse example. Noggings, particularly those at mid height of the studs, serve an important structural function and are not there just to provide fixing for the wall linings. They also provide restraint against buckling of the studs in the plane of the wall. The noggings illustrated in Figures 11 and 12 could not possibly perform this duty.

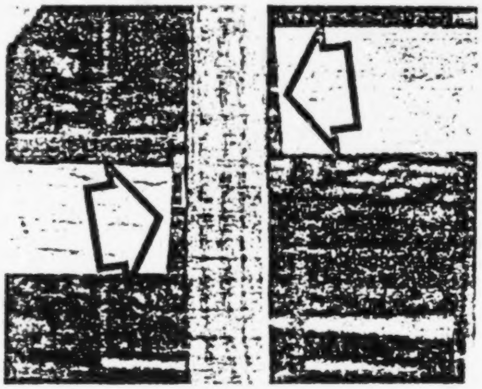

Figure 11 This stud is not going to get much help from the noggings.

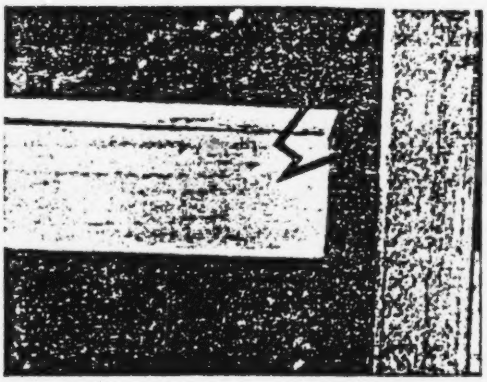

Figure 12 For all the good it can do this nogging might as well not be there!

Figure 13 is another example of making do. Here the two pieces of top plate have been joined with a piece of galvanized iron plate. Such a joint, particularly with the nailing used, one nail on one side, two on the other, serves no useful purpose. Indeed it has not even helped to keep the plates in the same line.

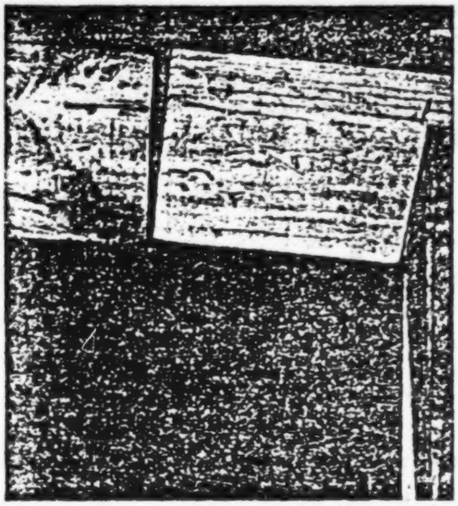

Figure 13 The galvanized plate linking the pieces of top plate is no more than a token gesture. It is difficult to see it performing any useful function.

A case of near enough being not good enough is shown is Figure 14. The header has been cut too short; it is virtually hanging on the nails at its ends instead of sitting on the base of the notches cut in the studs to receive it. Any roof load which happens to fall on the top plate would probably be transferred to ground through the architraves.

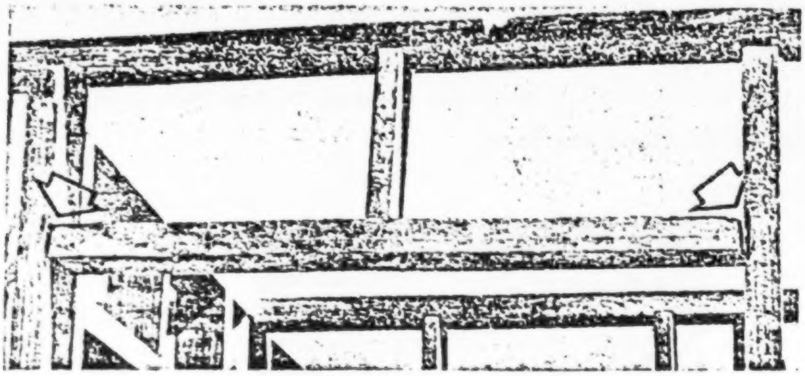

Figure 14 The header is more of a hanging beam - hanging on the nails.

7.5 Props

Figure 15 shows an underpurlin prop at an angle of 70° to the vertical. At this angle it cannot do its job effectively. The builder seems to have had second thoughts as the end of the underpurlin has been packed up with mortar from the brickwork. When the mortar has set it will probably be much more effective than the timber prop.

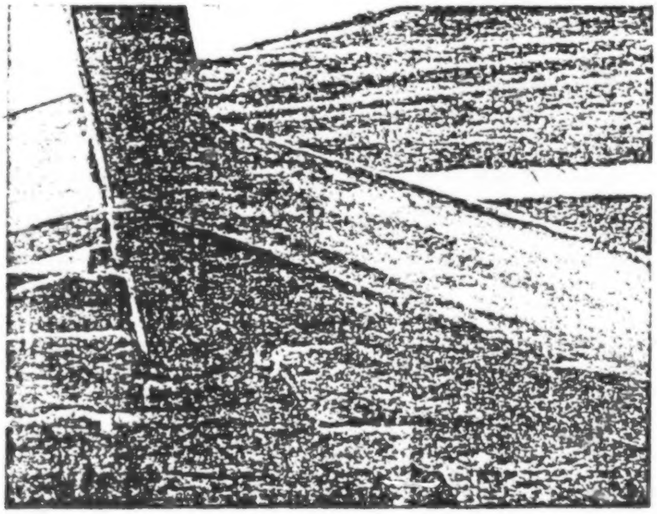

Figure 15 The mortar is doing more to support this underpurlin than the carefully notched strut.

7.6 Party Walls

The type of unit construction illustrated in Figure 16 is becoming increasingly common. Building regulations require a brick party wall between the units. The builder of these units has not only made sure that the units were divided according to regulations but were seen to be divided! Because no allowance or certainly a totally inadequate allowance has been made for the roof members (whether rafters or trusses) sagging as they inevitably will do, the roof battens are now bearing on the party walls. The consequent effect which is accentuated by the long length of unbroken roof is hardly pleasing to the eye. A similar effect results when an extra rafter is placed each side of the party wall as in Figure 17. These extra rafters, at much closer centres than the common rafters, make the roof much stiffer at the party wall because each of these rafters is more lightly loaded than the rest. They will therefore not sag as much and the roof will show a wave over the top of the party wall. A uniform spacing of all the rafters would avoid this problem.

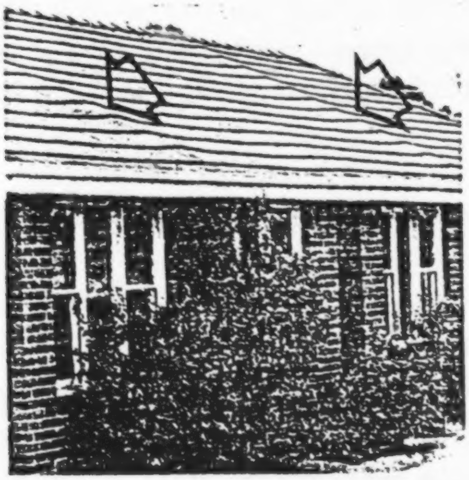

Figure 16 The whole appearance of these units has been spoilt because insufficient clearance was provided between the party walls and the roof system

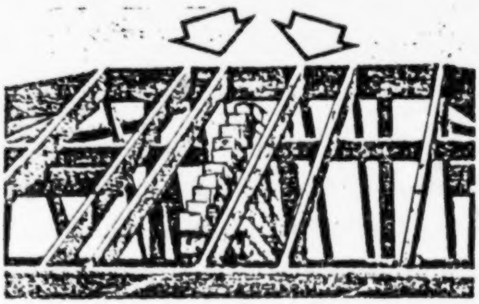

Figure 17 A wave in the roofline can be expected over the party wall because of the greater rigidity of the doubled-up rafters in this area

Most of the sag in the roof members takes place in the first 12 months after the tiles have been laid. So in these examples, which are typical of many that can be seen around Melbourne, the overall fresh and clean appearance of new building has been quickly depreciated by the poor appearance of the roofline.

8. THERMAL CONSIDERATIONS

The thermal requirements of a building vary considerably for different climates and consequently thermal performance of any particular type of construction may be satisfactory in one location and completely hopeless in another.

A recent document published by CSIRO Division of Building Research entitled 'A Comparison of Thermal Performance of Heavyweight and Lightweight Construction in Australian Dwellings' by P J Walsh, I A Gurr and E R Ballantyne, gives details of the performance of various types of dwellings in a wide range of climatic conditions. In general, it appears that the use of insulating material is of more significance than whether or not a wall is of cavity brick construction or brick veneer.

9. ACOUSTIC CONSIDERATIONS

In general, the acoustic performance of a domestic type of structure is determined by the number of doors, windows, etc. in the structure. However, Appendix A of AS 2021 Code of Practice for Building Siting and Construction Against Aircraft Noise Intrusion, gives acoustic performance information for several different types of construction. Other information is available in Australian Experimental Building Station Technical Study 48 'Airbourne Sound Transmission Through Elements of Buildings' by E I Weston, M A Burgess and J A Whitlock.

10. COST OF CONSTRUCTION

On-site costs account for about 75% of the selling price for moderate sized Australian houses. Of this amount, about two-thirds is due to materials and one-third due to labour. About 60% of the on-site cost is spent on the actual structure of the house.

Mr W D Woodhead of CSIRO Division of Building Research discusses the productivity of various methods of building different parts of a frame in a paper entitled 'Achievable Improvements in Housebuilding Productivity' which was presented at the Housing Industry Association Twelfth National Convention in April 1977. Several tables from that paper, which are self-explanatory, are reproduced here.

TABLE 4
DISTRIBUTION OF HOUSING CONSTRUCTION COST

Foundations and floor	10-15%	Structure	60%
Walls	25-30%		
Roof	18-23%		
Fixings, finishes and extras	approx. 25%	Fittings finishes services extras	40%
Services	approx. 15%		

TABLE 5
PERCENTAGE OF ON-SITE TIME FOR ELEMENTS OF THE HOUSE CONSTRUCTION PROCESS

Element	Percentage of on-site time			
	Brick veneer house on Timber sub-floor, 'economy' quality finishes		Cavity brick house on concrete slab, 'medium' quality finishes	
Sub-floor and floor	13%		7%	
Walls	24%		29%	
Roof	13%		16%	
Total structures	---	50%	---	52%
Services: Plumbing	10%		5%	
Electrical	2%		3%	
	---	12%	---	8%
Finishes: Timber	17%		10%	
Paint	14%		13%	
Tiles, etc.	2%		9%	
	---	33%	---	32%
Extras: Concreting, fence, cleaning		5%		8%
Total house		100%		100%

TABLE 6
SITE PREPARATION FOR DOMESTIC CONCRETE SLABS
Productivity on-site man hours with machine

Site type	Productivity Man hours/100 m²
Flat size slope < 1:25	3-6
Medium slop 1:25 to 1:10	6-12
Difficult sloping sites, substantial cut and fill, perhaps rock	12-30+

TABLE 7
PRODUCTIVITY FOR DOMESTIC SLAB CONSTRUCTION

Slab type	Productivity Man hours/100 m²
Light raft	40-55
Light or medium raft with internal beams	55-80
Suspended raft	80+

TABLE 8
PRODUCTIVITY FOR CONCRETE SLABS AND CONVENTIONAL SUB-FLOORS AT GROUND LEVEL

	Productivity Man hours per 100 m²	
Site conditions	Concrete slab	Conventional sub-floor
'Easy'	40 to 60	60 to 70
'Less easy'	60 to 90	70 to 80
'Difficult'	90 to 130+	80 to 100

TABLE 9
PRODUCTIVITY LEVELS FOR LAYING TIMBER STRIP AND SHEET FLOORS

Location	Type of house and floor	Flooring	Product. man hours per 100 m²
Vic	Ground level, laid room by room	4" Hardwood	16
Qld	Platform, part ground level	3" Hardwood	18
Qld	High set, platform	Plywood	16
Vic	Ground level, platform	Plywood	10
Vic	Ground level, plateford	Plywood	8

TABLE 10
PRODUCTIVITY FOR WALL FRAME CONSTRUCTION

Location	Type	Number	Man hours per 100 lineal metre		
			Factory(3)	Site	Total
Victoria	Site-cut softwood	1	Not applic.	47	47
"	" hardwood	3	"	40, 50, 52	40-52
Queensland	" " (elevated house)	1	"	84	84
New South Wales	Precut timbers: hardwood	2	22	23, 27	45-49
Queensland	" " softwood	1	Not avail.	40	-
Aust. Cap. Terr.	Preassembled frame: softwood	1	24	24	48
Victoria	System with mainly steel studs, timber plates, and noggings(1)	3	Not applic.	23, 23, 24	23
Victoria	System welded steel(2) and timber top plate	3	26-35	9, 12, 14	35-49

(1) Steel studs delivered to site complete with clips for fixing; timber dressed but not cut to length
(2) Rolled steel channel delivered to factory in set lengths
(3) Factory time does not include handling

TABLE 11
PRODUCTIVITY AND DISTRIBUTION OF TIME FOR THE SUB-ELEMENTS OF 12 ROOFS
(excluding ceilings)

Sub-element	Average productivity man hours per 100 m² covered area	Per cent. of total roof time
Roof frame	19	29
Fascia	4	6
Plumbing (ex. downpipes)	5	8
Battens and tiles	18	27
Eaves (frame and lining)	10	15
Gables (frame and trim)	10	15
Total roof	66	100%

TABLE 12
PRODUCTIVITY LEVELS FOR 12 ROOF FRAMES
(excludes the frame for the gable end where applicable)

No.	Location	Method of construction	Roof type(1)	Area to fascia m2	Productivity man hours per 100 m2 of covered area
1	Vic	Site-cut hardwood	3 hips	147	18
2	"	" "	"	165	23
3	"	" "	"	165	29
4	Qld	Site-cut hardwood (house raised 6 ft. off the ground)	3 gambrel gables	177	25
				Average 24	
5	ACT	Trusses(2)	2 plain gables	129	8
6	SA	Trusses(2)(3)	" "	130	15
7	Vic	Trusses + intermediate joists	" "	164	23
8	WA	Trusses + site-cut pieces(4)	L-shape, 2 gambrel gables	240	10
9	Qld	Trusses + ceiling battens (high set house)	2 plain gables	101	31(5)
				Average 14	
10	NSW	Precut hardwood timbers	hips	121	12
11	"	" " "	"	103	11
12	Qld	Precut softwood timbers	2 hips	174	22
				Average 15	

(1) All roofs designed for tiles except No. 9 clad with corrugated iron; fascia not included
(2) Time for installation of ceiling battens estimated at 6 man-hours
(3) Fixed by complex brackets
(4) 38% of time due to ceiling battens
(5) This was an a-typical roof constructed by day labour and was not included in the average

11. REFERENCS

1. Standards Association of Australia (1979). AS 1684-1979 Timber Framing Code.

2. Standards Association of Australia (19). AS 1720 Timber Engineering Code.

3. Standards Association of Australia (1971). AS 1170 Part 1-1979 SAA Loading Code, Part 1 Dead and Live Loads.

4. Standards Association of Australia (1980). AS 1170 Part 2-1980 SAA Loading Code, Part 2 Wind Forces.

5. Walsh, P.F., and Ballantyne, E.R. (19). A Comparison of Thermal Performance of Heavyweight and Lightweight Construction in Australian Dwellings. CSIRO Division of Building Research Technical Paper (Second Series) No. 44.

6. Standards Association of Australia (19). AS 2021 Code of Practice for Buildding Siting and Construction Against Aircraft Noise Intrusion.

7. Weston, E.J., Burgess, M.A., and Whitlock, J.A. (19). Airborne Sound Transmission Through Elements of Buildings. Australian Experimental Building Station Technical Study 48.

8. Woodhead, W.D. (1977). Achievable Improvements in House Building Productivity. Paper presented in Twelveth National Convention of Housing Industry Association, April.

CASE STUDY OF TIMBER CONSTRUCTION - KENYA HOTEL

Peter A. Campbell[1]

INTRODUCTION

This case study is based upon works carried out on a 200 bed tourist hotel on a Kenya Coast completed in 1974. At the time the writer was in practice as a consulting engineer and was responsible for all timber, reinforced concrete and steel structural works on site. The purpose of this note is to illustrate some of the problems that can occur and solutions that may be found in the use of timber.

The clients were a German tourist organisation and they wanted the hotel to have a strong architectural character and this was something that the architects and the writer had been able to achieve in several hotels, partly through a wider use of timber than was normally the case in Kenya.

TECHNICAL BACKGROUND

The hotel was to be built on top of a low cliff a few metres from the sea and so was exposed to sea breezes all the year around. The equilibrium moisture content was 16% to 18% and so was damp enough for the so-called 'drywood' termites, Cryptomeres spp. which were very active all along the coast. Subterranean termites were also present as the soils were predominantly sandy and in the prevailing moist conditions, were active through the year. The margins against decay were low and any minor leaks, etc. would dry out slowly and encourage decay. However the presence of salt in the air would to some extent counter this. This same salt and ambient temperatures of around 30°C made rust a very significant problem in the design of both steel work and reinforced concrete and stimulated the use of timber.

The general construction was to be of reinforced concrete with walling of exposed site-cut coral blocks. The roof over the whole hotel complex was of palm thatch on mangrove poles following traditional construction. Under this in the bedrooms and kitchen were concrete slabs (for fire reasons) and elsewhere over the larger spans a combination of timber and concrete beams supported the roofs. The general construction was limited to three floors and the hotel with bungalows sprawled through an open forest with many separate roofs.

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The main timbers available for construction were Cypress (C. lusitanica), a locally grown exotic conifer, and mangroves (Ceriops tagal and Rhizophera mucronata) and for large poles, Eucalyptus saligna. There were no seasoning kilns and so sawn timber for all purposes was air seasoned and much of this, not very well seasoned. The problems with an EMC of 16 to 18% were not too bad, but in the air-conditioned portions of the hotel with a much lower EMC, some troubles were expected. Pressure treatment facilities were available for the cypress and saligna which were treated up-country and shipped down 600 km to the coast. But this facility was far too expensive to consider for mangroves. The grading rules were very poor having been drawn up by the Forest Department without reference to engineering requirements. This was by-passed by writing performance specifications which were included in the contract documents and so over-rode the government imposed specifications (Campbell 1971a). Strength values were available for cypress and saligna (Campbell & Malde 1971b). The saligna values were not the same as those in use in Australia or South Africa where the same species was grown as an exotic, as growth conditions were different.

Strength values for the mangroves were not available in the year or so which was available between initial design brief and construction start, a number of poles were collected from along the coast by the Forest Department and tested at the University of Nairobi by the writer (Campbell 1973). This provided the necessary strength data.

ROOF COVERINGS

As noted above, these were of mangrove pole-framing under palm thatch made locally. These roofs had a life of about ten years after which a maintenance gang was sent through to tie up the poles and rethatch. Traditionally, these poles were tied together with 'miaa', a string made from local dwarf palm. Sisal string was substituted to make rather stronger joints and the writer spent some time teaching the traditional roof makers how to tie better knots. The mangrove poles in roofs, though described as 'perishable' in the 'durability' classification, had a reputation over many generations, of long life in roof construction. The only factor which had changed had been the introduction of Cryptotermes in about 1940 and which thirty years later had become widespread along the coast. All enquiries pointed to these mangroves being resistant to this insect. They were known also to be resistant to subterranean termites when used above ground.

As poles they came with sapwood, but traditional post-felling treatment included soaking in seawater for some weeks and there was no evidence of sapwood being attacked by anything. One minor innovation was the introduction of precast concrete shoes which were set in concrete members for attaching mangrove pole structures.

MAJOR ROOF STRUCTURES

There were several areas which were much too large to be covered with mangrove poles and some of these were given primary support with treated saligna poles up to 400 mm diameter. These were cut to length before treatment at the plant some 800 km away. Erection of these poles was carried out by a steel erector using gin poles. Cranes were not available and even had they been, the contractors organisation was not geared to the rapid erection which would have been concommittant. The ends of the poles were cut to receive steel fixings in many cases and this, of course, went through the protective coating. In addition, there was some end splitting though not as much as there would have been in a dry climate and this was reduced to some extent by slotting and ring grooving the ends of the poles. The ends were treated with protective chemicals which at the time appeared to be valid, but in retrospect, were probably a waste of time.

WOODEN DOWELS

One of the major costs of trusses in East Africa had been bolts and some work had been done looking at dowels as an alternative. On this project, rust on steelwork was inevitable, unless very heavily galvanised and well maintained. In early discussions with the architects, wooden dowels were suggested as likely to be much cheaper and more durable. This was accepted and incorporated into the idiom of the architecture especially in handrails. Some 10 000 dowels, mostly hand made from mangrove off cuts were used. In one structure about 13 m diameter with a centre pole, the rafters were attached to the centre pole with wooden dowels only and these were from proper dowels made from Iroko.

SEASONING

Bad seasoning was one of the main reasons why architects in Kenya disliked timber. The seasoning defects were irritating and expensive in replacement costs. The timber industry had no building experience and so no understanding of the architects' problems. Carpenters and joiners and technicians were not taught seasoning because the English syllabuses did not include this. The response to this was to get builders to sticker - stack timber on site early where theft was not a problem. 25 mm structural timbers would partially season sufficiently so that careful detailing could avoid warping problems.

Structural members were made up from 25 mm boards by vertical laminations with nails. This is not only solved virtually all the seasoning warp that had previously been such an annoyance but also led to greater structural stability as continuous beams became possible. With joinery timbers the problem was more difficult. An attempt was made to get joiners to use dowelled joints in place of mortice and tenon joints which are very sensitive to movement. Timbers with low shrinkage - less than 1% tangential - were used where possible. Perhaps the main innovation was the requirement that contractors have on site a moisture meter and a requirement to this effect was put into contract documents. This concentrated attention on the need for proper seasoning and contractors fed this back to suppliers with some improvement in quality. It was also found necessary to give gentle briefings to furniture designers and architects on how to detail for movement. It was not considered proper for engineers to instruct architects on their detailing so some tact was required and led to far fewer failures of table tops, etc. and more confidence in the use of timber.

MANGROVE CONCRETE SLAB SUPPORTS

Traditional upper storey floors were made of mangrove poles placed close together, packed with coral lumps and then topped with a lime plaster floor finish. Such a ceiling certainly had character as did the inevitable nonplanar floor. It was decided to reproduce this effect and so a study was made of the various interactions between mangrove poles and reinforced concrete. The final design had mangrove poles placed touching each other cross the short span. These were propped during construction to avoid the characteristically large deflections of the traditional floor. Over these, was laid polyethelene sheeting followed by reinforcement and concrete. After casting, the plastic was burnt off and the soffit oiled. The finished floor relied entirely on the steel reinforcement as the stiffness of the poles was much lower than that of the reinforced concrete. The mode of failure of traditional roofs was through decay of the pole ends where they were buried in the walls. In this project the ends of the poles were left exposed on a ledge. The rooms were air conditioned so the control of insects, etc. which would be naturally attracted to holes around the exposed ends, could be controlled.

TIMBER JOINERY

The architectural profession has to cover a very wide variety of materials and is traditionally weak on basic science and so engineers may often find themselves advising architects on materials. In this case the main joinery timber was Mvule (Chlorophora excelsa, Iroko in West Africa) and this was resistant to everything and an excellent joinery timber by international standards. This all came from Uganda and at the time there were some slight political problems as a result of which the mvule was not coming out. It was realised early that significant delays could be anticipated and so a start was made looking for an alternative.

Several species which appeared suitable were found by the writer through the literature and samples obtained. The Forest Department told the architects that there was plenty available and so the architects relaxed. The writer checked up and found that there was indeed an abundance of the timber available as trees. It could not be extracted until after the next rainy season and after conversion and air seasoning it would be at least 9 months before the material could get into the joinery shops and this was far too late. About this time the contractor solved the problem by transporting mvule in from Tanzania which did not have very much and had banned its export to soft currency countries such as Kenya. The point of this comment is that architects (and many engineers) on the one hand and foresters on the other, (and millers) have very little idea of each others problems and communication difficulties are to be anticipated.

THE CONTRACTOR

In some countries there are very many sub-contractors available and the skill of the main contractor is in managing the many 'subbies'. In less advanced areas, and this included Kenya, there were very few sub-contractors and so the main contractor was engaging and responsible for supervising, in detail, a very much wider range of trade skills than would his counterpart in, say, Australia. This meant that the main contractor may have to have more technical skills if working in a developing country than in an industrialised country. Where special care or site craft skills are necessary to cope with variable products, such as timber in Kenya, this requires some thought by designers. (Site craft skills are those which cover labours and products which cannot be easily described in specifications as there are no quality assurance specifications available and so supervision is mainly subjective and not objective). One approach to this is to put in more explicit specifications and this may be more satisfying to the specification writer than the contractor if the latter does not read English too well. What may happen in practice is that designers and quantity surveyors give much more assistance to the contractor even advising him when he puts in ridiculously low costs. In return the contractor responded to comments on quality which were not explicitly covered in the specifications because of a lack of Standards. Where there was some innovation, this was explained in detail to the builder very carefully whilst tendering and again before construction.

In this project and a number of others where there was a much higher timber content than was normally the case in East Africa, much time was spent with contractors teaching them about timber, i.e. how to sticker stack and an innovative designer in a developing country should expect to have to do this.

CONCLUSION

There was no traditional building culture which related to the functions or scale of modern buildings in Kenya. Such is not the case in S.E. Asia where there are very rich and ancient cultures often expressed through buildings. Today many of the materials originally used to construct these buildings are becoming difficult to obtain or very expensive and there is sometimes a shift away from traditional building idioms on this account. One of the challenges for designers today is to find ways of using more economical materials, such as new species of timber, to maintain and improve traditional cultures as expressed through buildings. Anyone can join the concrete, corrugated iron and coke movement; it takes more skill and dedication to also be able to design in timber. It is also more satisfying.

REFERENCES

Campbell, P.A. 1971a 'Performance Specifications for the Quality Control of Timber for use in Housing in Developing Countries'. World Consultation on the use of Wood in Housing, Vancouver 1971.

Campbell, P.A. and Malde, K. 1971b 'Timber for Building in Tanzania'. Tanzania Forest Department 1971.

Campbell, P.A. 1973 'Mangrove Poles for Construction'. Occasional Paper No. 7, Department of Civil Engineering, University of Nairobi 1971.

CASE STUDY OF TIMBER CONSTRUCTION - NEW ZEALAND

G. B. Walford[1]

1. INTRODUCTION

It is the intention of these notes to describe three examples of timber construction with the object to show that it is possible to build timber structures economically and achieve a result that is both functionally and aesthetically pleasing. It should be emphasised that a structure is considered economical only in relation to the availability and cost of materials and skills at a certain place and time. Another factor that makes comparison difficult is the social or prestige value associated with a building. For instance in Auckland, New Zealand, in 1982 the old Customhouse building of brick and timber construction was strengthened and refurbished at a cost of $3M, about the same amount necessary to replace the building with one with six to eight times the earthquake resistance, i.e. built to current structural standards.

The examples presented are:

(a) a farm building incorporating timber portal frames,

(b) a single storey 4526 m^2 warehouse of nailed plywood and sawn timber construction,

(c) a four storey composite timber and reinforced concrete office building of 3900 m^2 together with a single storey trading building of 4200 m^2.

2. FARM BUILDING

2.1 Structural system

The "HB system of timber construction" is described briefly in the "Timber Construction Manual" produced by the Canadian Institute of Timber Construction in 1959. It consists of structural beams and frames of I cross section built up by nailing and/or gluing timber flanges onto a web made of two layers of boards placed at right angles to each other and at 45 degrees to the axis of the members. Figure 1 shows details of a frame and an indication of the variety of shapes that have been built in this system. Examples are also illustrated in the "Timber Engineering Design Handbook" by Pearson, Kloot and Boyd, published in 1958. Examples have also appeared in reports originating from the Forest Research Institute in Dehra Dun, India.

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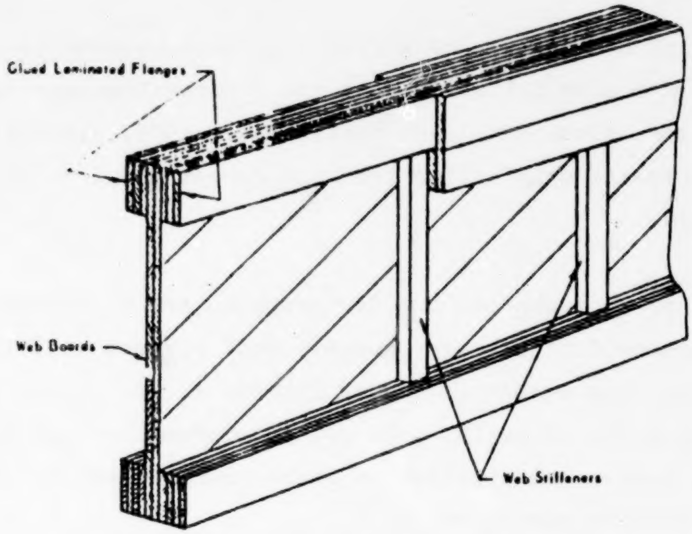
The system is by no means new but few examples are to be seen nowadays. The reasons for this are probably that plywood will provide a better web material, the system is labour intensive, and if the flange-to-web connection is nailed then creep deformations can be large. Nevertheless, the system has application where sawn timber, nails and labour are the available resources.

2.2 Woolshed design

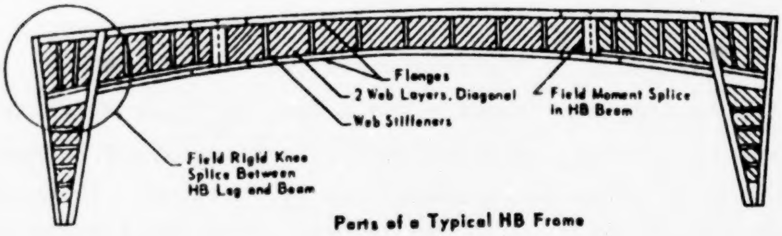
The farm building shown in Figure 2 was a two storey woolshed. This design allows for holding pens beneath and a loading stage which is a convenient height for loading bales of wool onto a truck. The frames were built entirely of 25 mm thick timber and were nailed. Although the design specified 100 mm nails at 100 mm centres along the flanges and twice this density in the knee region, far fewer nails were actually driven. The construction procedure for the frames requires a flat working surface. In this case the framing for the woolshed floor was first erected and this served as a working surface.

It was found much easier to pour the concrete footings at their correct positions but at whatever height the ground level dictated than to level the site first. The length and taper of the portal legs were varied to allow for the differing levels. The resulting differences in taper were not noticeable.

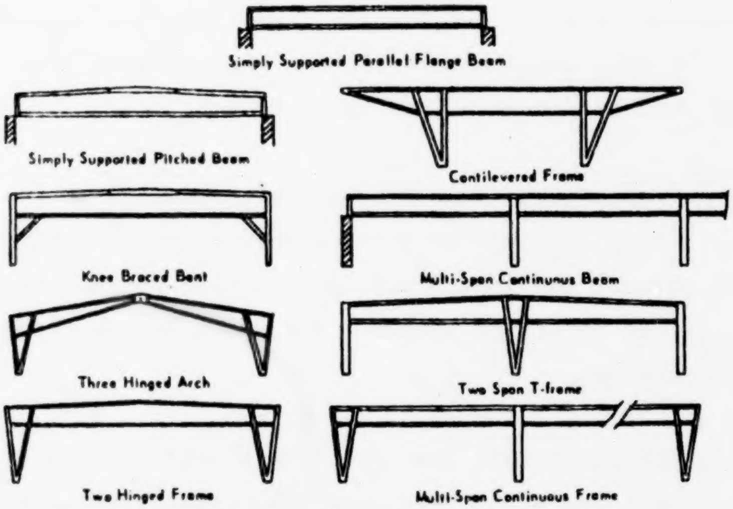
An alternative design in this case would have used trusses spanning 8.4 m from wall to wall and would have used less timber but would have required more attention to bracing against wind loads. The particular advantage of the portal was that roosting spaces for birds could be eliminated and 3 m headroom was available for operation of the woolpress.


Glued Laminated Flanges
Web Boards
Web Stiffeners

Typical Section of an HB Member


Flanges
2 Web Layers, Diagonal
Web Stiffeners
Field Moment Splice in HB Beam
Field Rigid Knee Splice Between HB Leg and Beam

Parts of a Typical HB Frame


Simply Supported Parallel Flange Beam
Simply Supported Pitched Beam
Cantilevered Frame
Knee Braced Bent
Multi-Span Continunus Beam
Three Hinged Arch
Two Span T-frame
Two Hinged Frame
Multi-Span Continuous Frame

Typical Structural Forms in which HB Members may be Arranged

FIG. 1 "HB" SYSTEM OF TIMBER CONSTRUCTION

Analysis and design of these frames is simple:

(a) Assume a span to depth ratio of about 10:1. This determines the depth at midspan of a beam or at the knee of a portal.

(b) Calculate flange sizes from the direct compression and tension stresses induced by bending moments.

(c) Resolve the shear forces at 45° to obtain forces in the web members. Usually these will be very lightly stressed.

(d) Resolve the forces in the web members parallel to the flanges and calculate the number of nails required from the allowable nail loads.

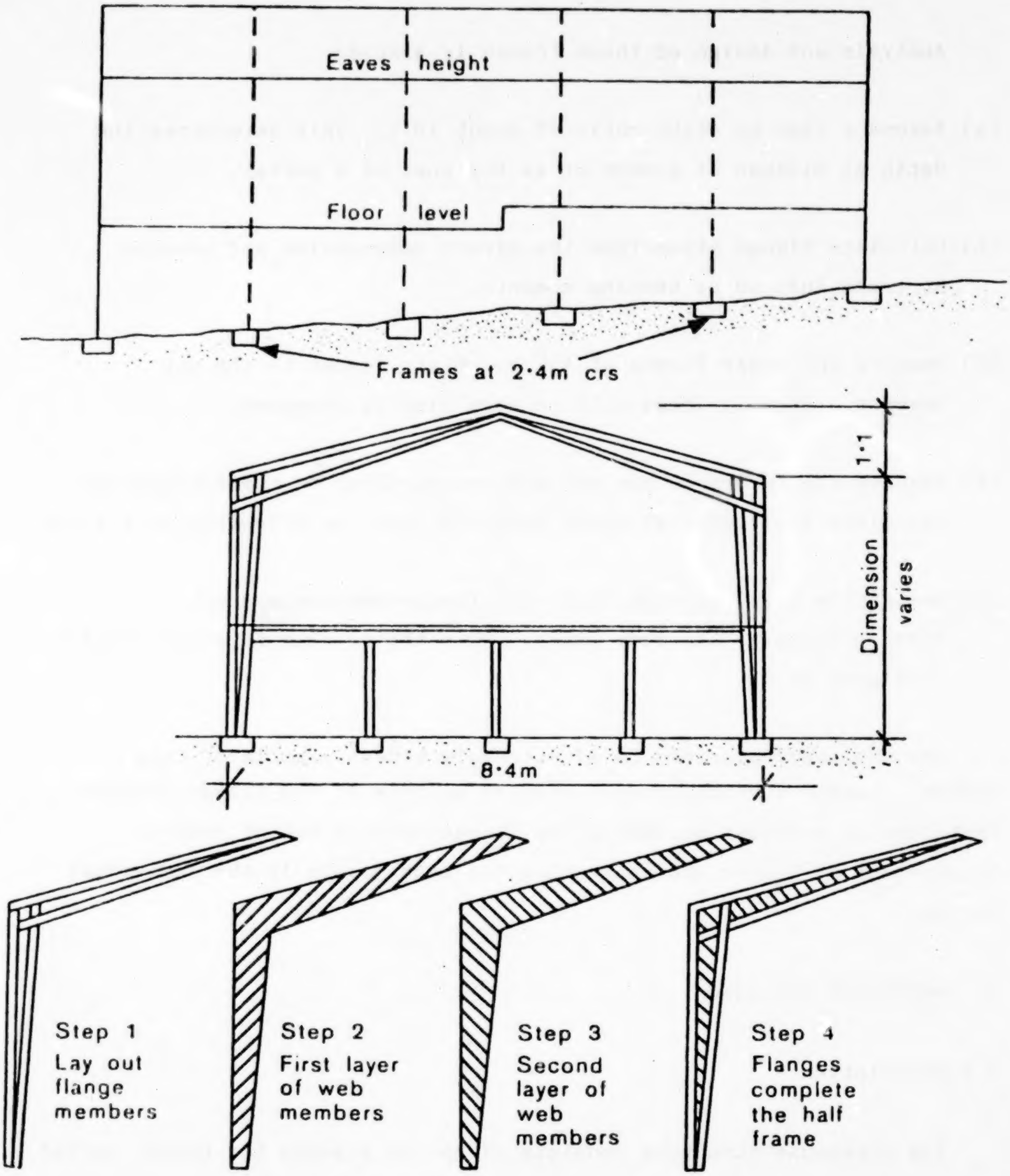
(e) Design the knee joint so that both flange members are not discontinuous at the same point. Note the details in steps 1 and 4 in Figure 2.

The woolshed described is an extremely modest example of this system. Larger structures will require splices in the flange members. These can be achieved by making the flange members out of several thicknesses of timber and staggering the butt joints in the individual layers.

3. WAREHOUSE BUILDING

3.1 Description

The warehouse structure consists of nailed plywood box beams, nailed laminated timber columns, and nailed plywood roof diaphragms and shear walls. Figures 3 and 4 show a plan and cross section of the building which is used for a paper warehousing and distribution operation. Paper is received in bulk form, then guillotined into standard sizes placed on pallets and stored in a rack system for retrieval and despatch. The incorporated office block has two floors of 600 m^2 each with a timber frame and plywood shear wall system for resisting lateral loads.


Eaves height
Floor level
Frames at 2·4m crs
1·1
Dimension varies
8·4m
Step 1
Lay out flange members
Step 2
First layer of web members
Step 3
Second layer of web members
Step 4
Flanges complete the half frame

Frame construction procedure

FIG. 2 WOOLSHED WITH TIMBER PORTAL FRAMES

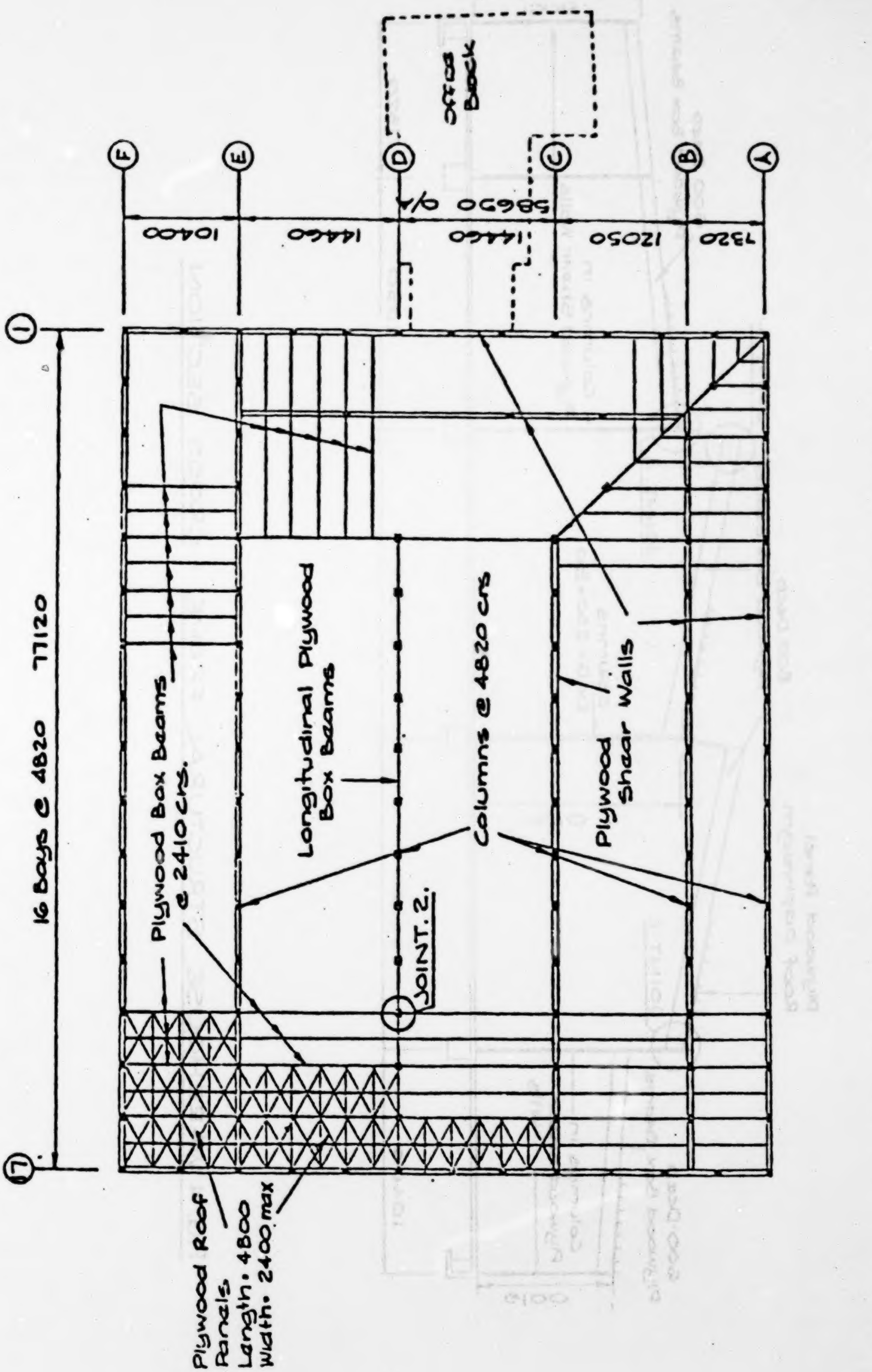

16 Bays @ 4820
77120
1
17
F
E
D
C
B
A
58650 o/A
10400
14460
14460
12050
7320
Office Block
Plywood Box Beams @ 2410 crs.
Longitudinal Plywood Box Beams
Columns @ 4820 crs
Plywood Shear Walls
JOINT. 2.
Plywood Roof Panels
Length • 4800
Width • 2400 max

FIG. 3 WAREHOUSE STRUCTURAL FRAME - PLAN

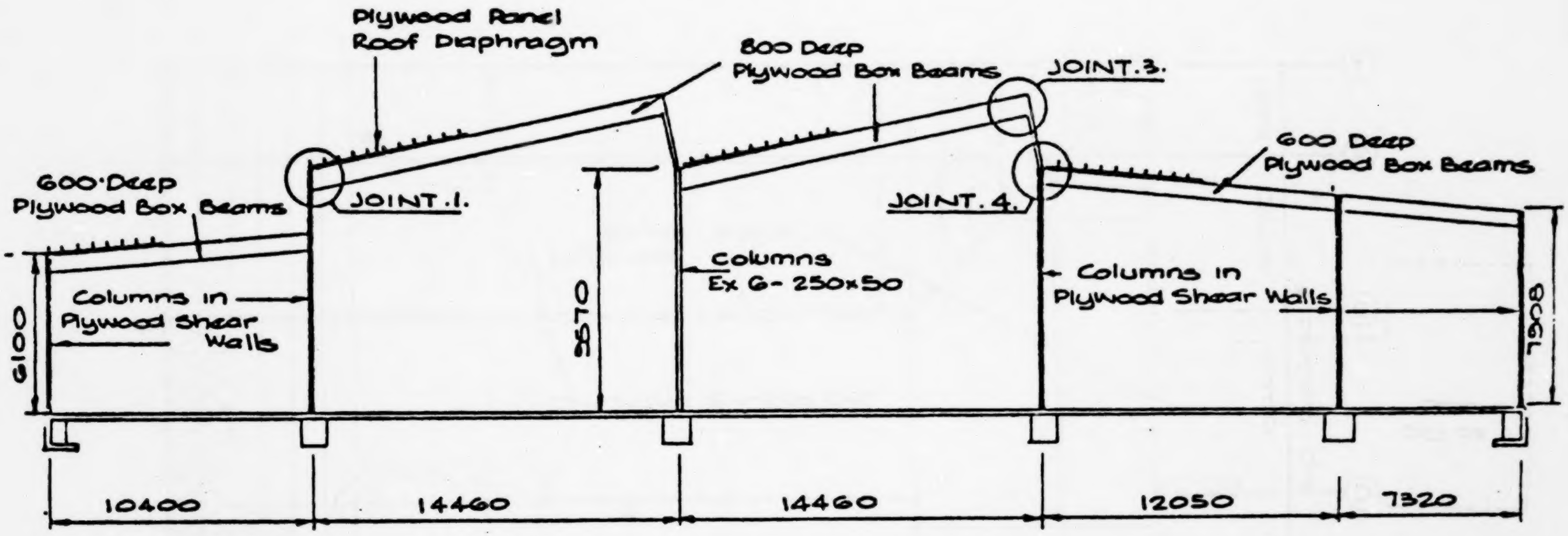

Plywood Panel Roof Diaphragm
800 Deep Plywood Box Beams
JOINT.3.
600 Deep Plywood Box Beams
600·Deep Plywood Box Beams
JOINT.1.
JOINT.4.
Columns Ex G- 250x50
Columns in Plywood Shear Walls
Columns in Plywood Shear Walls
6100
9570
7908
10400
14460
14460
12050
7320

FIG. 4 WAREHOUSE STRUCTURAL FRAME - CROSS SECTION

The design brief given stated that the building should be an economical form of timber construction and the functions of the various areas dictated the basic layout and roof shape. Walls divide the bulk storage area into two sections with a clear stacking height of 6 m and separate this area from the racking area with storage to a height of 8.3 m, and a further wall separates the cart dock and loading area. These internal walls, along with the exterior walls, when lined both sides make an ideal layout for a shear wall system. The roofline, although broken into several large panels, still provides a reasonable roof diaphragm system.

3.2 Alternatives

Various components were considered:

(a) Trusses - these were about 10% cheaper than equivalent nailed plywood box beams but were considered undesirable for this application because the accumulation of fire wood fibre would produce a fire hazard.

(b) Glulam beams = these were about 50% dearer than the plybox beams.

(c) Steel beams - these were about 35% dearer than the plybox beams and nearer 50% dearer when the added cost of fixings to the roof and columns were included.

3. Design Philosophy

Wall and ceiling linings that can also act as shear diaphragms is an ideal application for plywood. The additional nailings required costs little and allows the use of a pin jointed beam and column system to support gravity loads.

The selection of building modules had to cater for:

- the use of whole sheets for linings
- adequate tolerances in fitting together of components
- proportioning the various units for economy and ease of handling.

The first two criteria are easy to satisfy but the selection of spacings to give the best proportions requires considerable care - e.g. doubling the box beam spacing to 4820 would use the same amount of material in the beams but increase the purlin size from 125 x 50 to 200 or 250 x 50 and increasing the depth of the box beams would make them less stable.

Readily available materials were specified throughout. This is important as it is no use calling for materials that are scarce or of unreasonably high quality. Timber lengths were limited to 4800, and timber thicknesses to 50 mm. The plywood was 7.5 and 12.5 mm DD grade internally and 12.5 mm C-plugged D grade externally. The columns, beams, and roof panels were all prefabricated.

3.4 Components and Joints

The prefabricated components were all fully detailed in order to allow fabrication in a pre-cut factory directly from the drawings. Figures 5 and 6 show typical details.

The columns are assumed to be pin jointed top and bottom, and consist of up to six green gauged 250 x 50 mm pieces. Three rows of nails fix the laminations tightly together, and allow the transfer of gravity and wind uplift loads to the outer pieces for transfer to the baseplates which are shown in Figure 7. The longer columns on the exterior walls were strengthened with steel flitch plates nailed to their sides to resist bending loads due to wind. The horizontal girts are also 250 x 50 mm members and the plywood lining is nailed directly to these on both sides to form a shear resisting diaphragm.

The beams were of conventional plywood box beam construction with details shown in Figure 5. A problem arose with splitting in the chord 35 mm thick members as the design required 3.55 x 40 mm nails at 35 mm centres along both edges of each chord member. This was caused by the use of radiata of higher than normal density and was solved by using radiata from a different forest, increasing the chord thickness to 50 mm and spacing the nails at 40 mm.

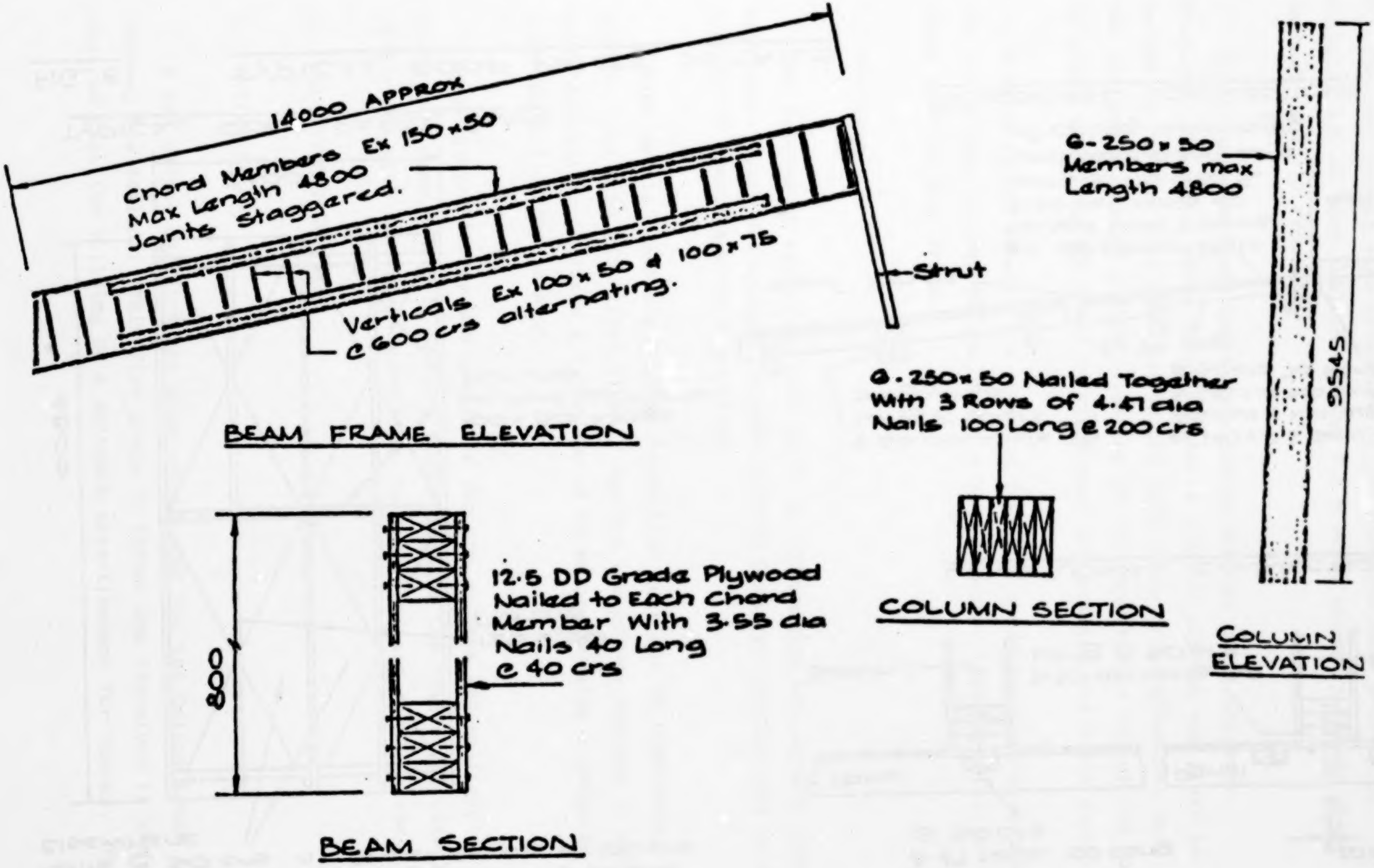
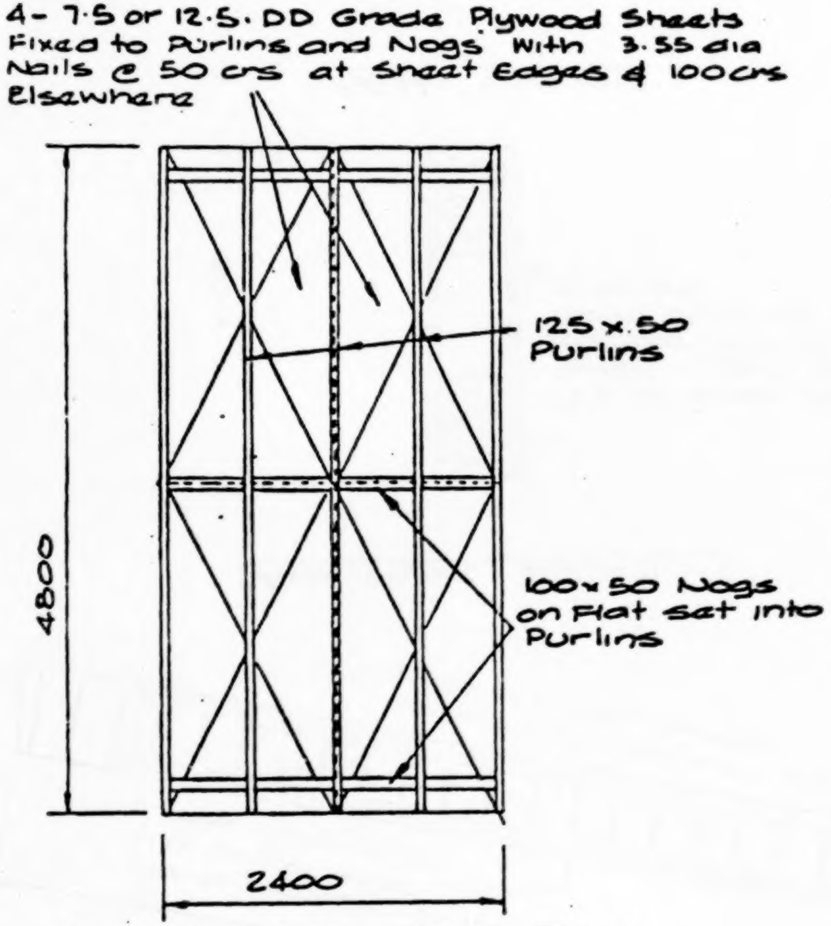
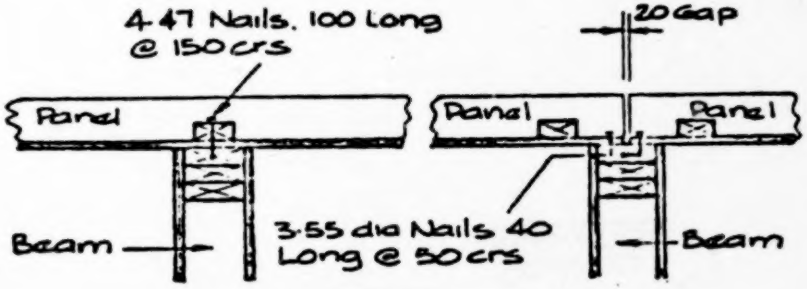

14000 APPROX
Chord Members Ex 150 x 50
Max Length 4800
Joints Staggered.
Verticals Ex 100 x 50 & 100 x 75
@ 600 crs alternating.
Strut
BEAM FRAME ELEVATION
800
12.5 DD Grade Plywood
Nailed to Each Chord
Member With 3.55 dia
Nails 40 Long
@ 40 crs
BEAM SECTION
6 - 250 x 50 Nailed Together
With 3 Rows of 4.47 dia
Nails 100 Long @ 200 crs
COLUMN SECTION
6 - 250 x 50
Members max
Length 4800
9545
COLUMN
ELEVATION

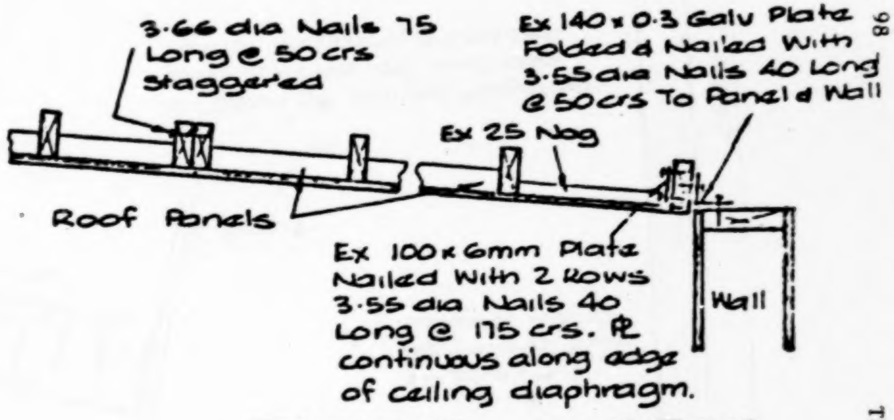
FIG. 5 TYPICAL BEAM & COLUMN CONSTRUCTION


4- 7·5 or 12·5· DD Grade Plywood Sheets Fixed to Purlins and Nogs With 3·55 dia Nails @ 50 crs at Sheet Edges & 100crs Elsewhere
125 x 50 Purlins
100 x 50 Nogs on Flat Set into Purlins
4800
2400

TYPICAL ROOF PANEL PLAN


4.47 Nails. 100 Long @ 150crs
20 Gap
Panel
Panel
Panel
Beam
3·55 dia Nails 40 Long @ 50crs
Beam

LONGITUDINAL CONNECTIONS


3·66 dia Nails 75 Long @ 50crs Staggered
Ex 140 x 0·3 Galv Plate Folded & Nailed With 3·55 dia Nails 40 Long @ 50crs To Panel & Wall
Ex 25 Nog
Roof Panels
Ex 100 x 6mm Plate Nailed With 2 Rows 3·55 dia Nails 40 Long @ 175 crs. PL continuous along edge of ceiling diaphragm.
Wall

TRANSVERSE CONNECTIONS

FIG. 6 TYPICAL ROOF PANEL DETAILS

The roof panels take advantage of the stressed skin principle, allowing 125 mm deep purlins to span 4.8 m where normally 200 mm deep members would be required.

Joints were designed to be very simple and use nails wherever possible. Figure 7 shows joint 1 details where a plywood end plate is nailed to the end of a beam and to the wall frame or longitudinal beam. The ceiling diaphragm connects the top of the beam to the top of the wall to avoid stressing the nails in withdrawal and pulling the joint apart.

Further joint details are shown in Figure 8 where the struts between the rooflights connect to the top of a beam or wall. Again, nails in shear provide the fixing.

3.5 Problems

The problem of splitting with nails at 10D spacing has already been mentioned. This was unusual because radiata pine can usually take nails down to 5D spacings without problems.

Joint detail 2 shows a typical connection in a row of beam-column joints. The beam end fitted tightly against the column whereas it would have been better to leave a tolerance of 10 mm and provide a ledge to rest the end of the beam on.

Green gauged timber was specified for the columns and wall girts. It would have been easier to use dry timber because the timber was actually partly dry with consequent variations in thickness and straightness.

4. MULTISTOREY BUILDING

4.1 Introduction

A new office building has been constructed for the Odlins Group in Petone, New Zealand. This firm deals in timber and therefore it was desired that the building be a suitable advertisement for timber. The

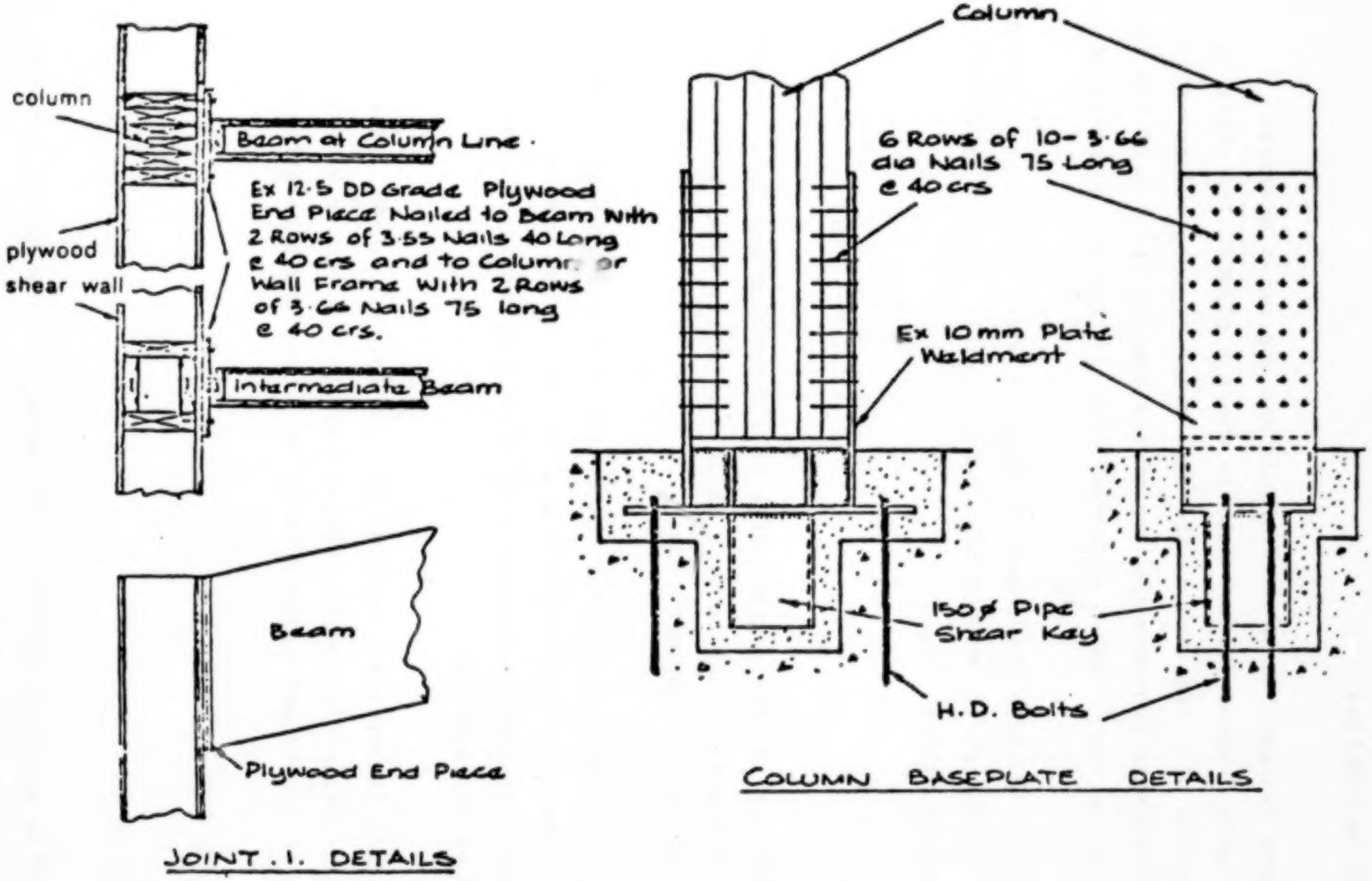
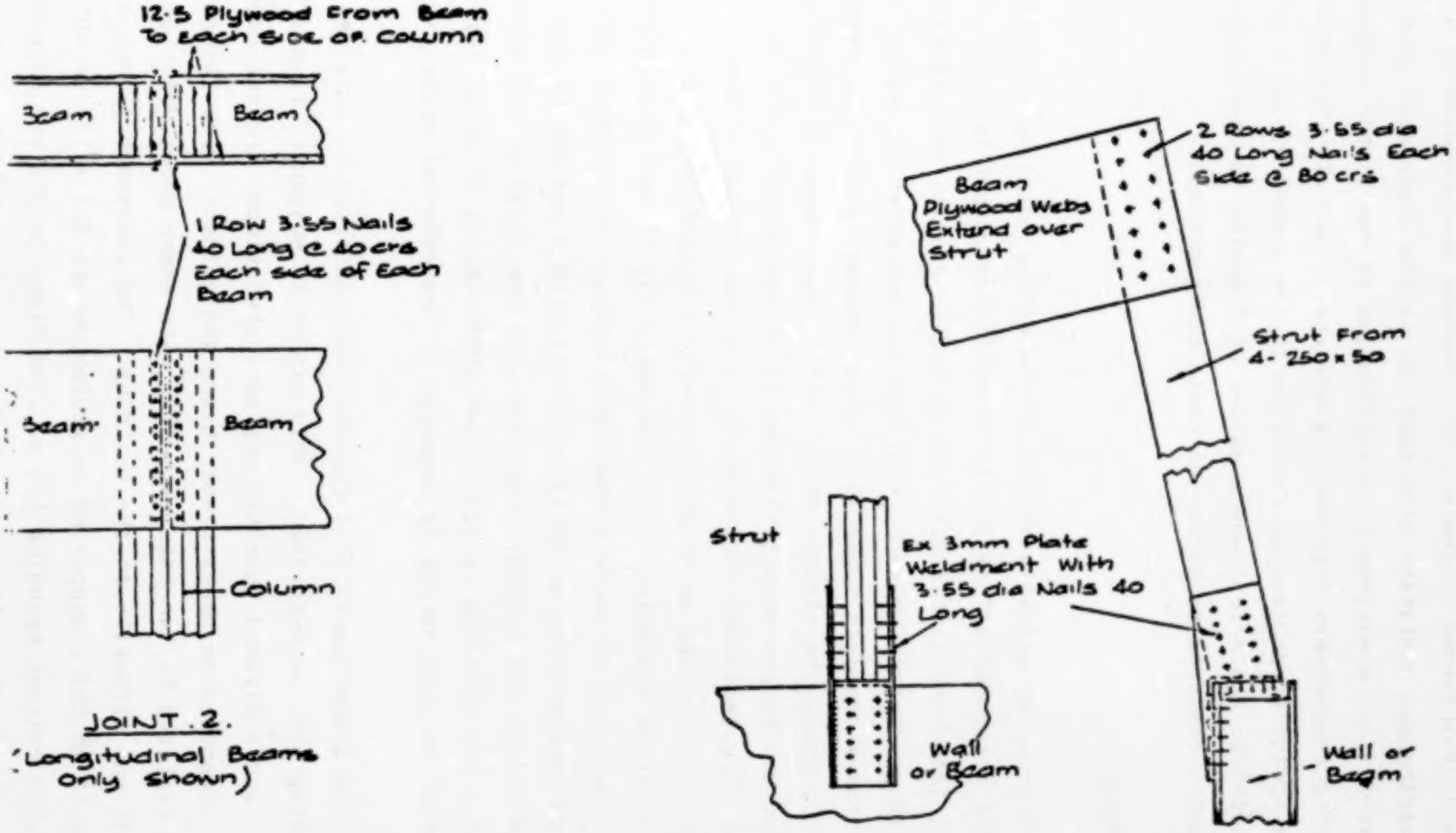

column
plywood
shear wall
Beam at Column Line
Ex 12·5 DD Grade Plywood End Piece Nailed to Beam With 2 Rows of 3·55 Nails 40 Long @ 40 crs and to Column or Wall Frame With 2 Rows of 3·66 Nails 75 long @ 40 crs.
Intermediate Beam
Beam
Plywood End Piece
JOINT . 1. DETAILS
Column
6 Rows of 10- 3·66 dia Nails 75 Long @ 40 crs
Ex 10mm Plate Weldment
150ø Pipe Shear Key
H.D. Bolts
COLUMN BASEPLATE DETAILS

FIG. 7 TYPICAL JOINT DETAILS


12·5 Plywood From Beam To Each Side of Column
Beam
Beam
1 Row 3·55 Nails 40 Long @ 40 crs Each side of Each Beam
Beam
Beam
Column
Beam Plywood Webs Extend over Strut
2 Rows 3·55 dia 40 Long Nails Each Side @ 80 crs
Strut From 4- 250 x 50
Strut
Ex 3mm Plate Weldment With 3·55 dia Nails 40 Long
Wall or Beam
Wall or Beam

JOINT. 2.
(Longitudinal Beams only shown)

JOINTS 3 & 4

FIG. 8 TYPICAL JOINT DETAILS

result was a design that capitalizes on the advantages of timber, avoids its disadvantages and ws 10% cheaper than the next most competitive alternative in reinforced concrete. This economy was achieved by a saving in earthquake resistant structure due to the lower dead load of timber construction, a minimum of specification of the timber components (i.e. avoiding unnecessary treatment, grade etc.), and with the provision of complete sprinkler protection from fire in the timber design. A further benefit to the client was a saving of 8 months in construction time (12 months compared to 20 for concrete construction).

4.2 Description

Figure 9 shows an artist's sketch of the office building and trading complex. The octagonal office block has reinforced concrete piles, foundations, floor and lift shafts with a timber gravity load resisting system of floors, beams, columns, roof trusses and exterior walls. The trade block has a concrete floor with timber columns supporting timber trusses and is braced by plywood shear walls. Only recently (1978) has such construction become permissible due to an amendment to N.Z.'s Standards on fire resistant construction to allow the use of timber construction in buildings up to four storeys in height provided a sprinkler system is installed. This amendment has allowed a significant increase in the scope of heavy timber construction, which previously was restricted to construction of small buildings with a maximum of two storeys, and brings the NZ code into line with the Canadian and United States codes. The building is also sited sufficiently clear of the site boundaries that no fire rating is required to the external walls.

Because the trade block is of conventional construction only the office building will be described. This has three suspended floors of 970 m^2 each, and a plywood sheathed prefabricated timber trussed roof principally supported on heavy timber beams and columns, with lateral loads being resisted by two reinforced concrete shear cores which enclose the stairwell and liftwells to the building. The suspended floors are glulam slabs 65 mm thick supported on glulam joists 405 x 144 or 405 x 219 mm at 2.5 m centres spanning 6.5 m. The floor joists are supported on glulam beam and column frames located at the outer perimeter wall, the

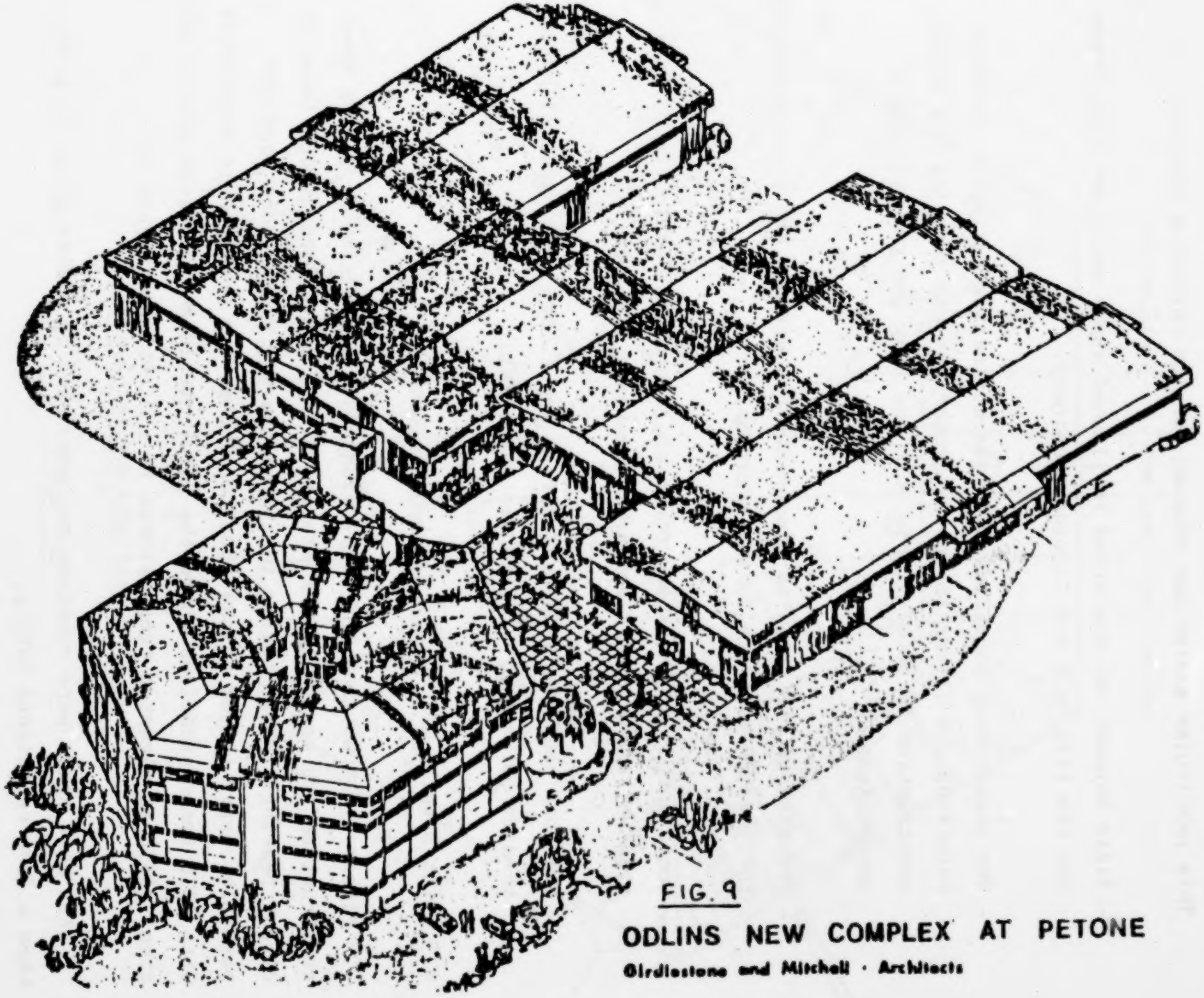


FIG. 9

ODLINS NEW COMPLEX AT PETONE

Girdlestone and Mitchell · Architects

inner perimeter wall and halfway between the two. Figure 10 shows typical beam, column, joist and floor dimensions.

This particular system was adopted for the following reasons:

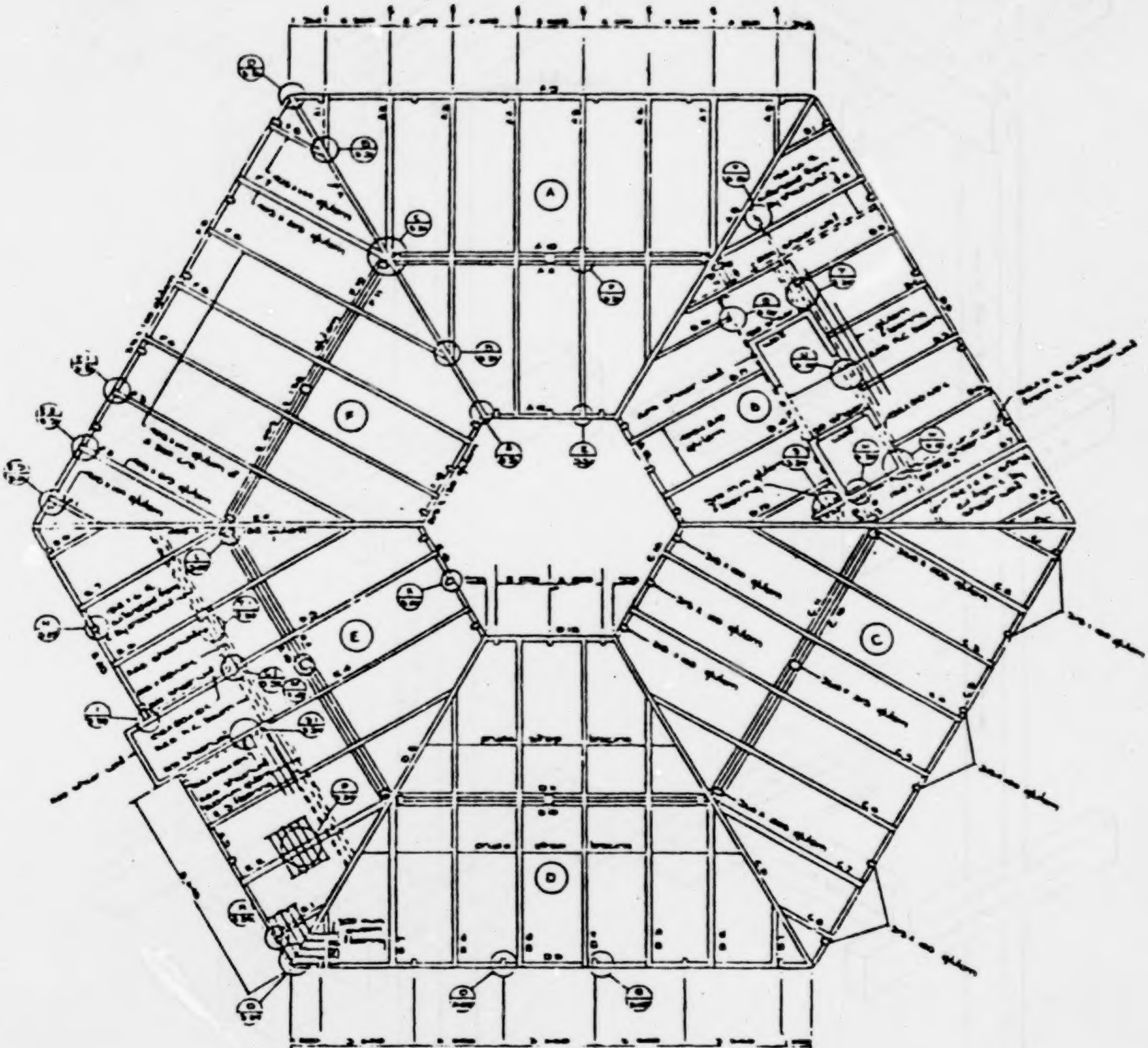
- Rigid support was preferred to plywood shear walls or steel frame for the lift well and lifting machinery.
- The shear cores localised the lateral loading resisting elements requiring the floors to act as diaphragms and enabling the timber construction to be of simple post and beam design with simple connections.
- The shear cores could be constructed in advance and provide support for the timber frame during erection.
- Construction of the shear cores would provide continuity of work on site while the glulam members were being prepared.
- The roof could be completed before the floors were laid, allowing them to be kept dry (which was essential).

4.3 Joint Details

Figure 11 shows that the beam-column joints are simple, giving easy construction and good fire resistance, with gravity loads being taken in direct bearing. Where timber to timber bearing connections were not possible, heavy (10 mm thick) steel brackets were provided in accordance with AITC recommendations where the joint should not collapse should the steel yield at elevated temperatures in a fire. See Figure 12.

All bolt heads were recessed and concealed by timber plugs to give them a fire resistance rating.

The column base fixing was achieved with steel dowels screwed into the end grain of the columns and grouted into ducts in the concrete after the frames had been erected and aligned.



FIRST & SECOND FLOOR FRAMING PLAN

FIG. 10

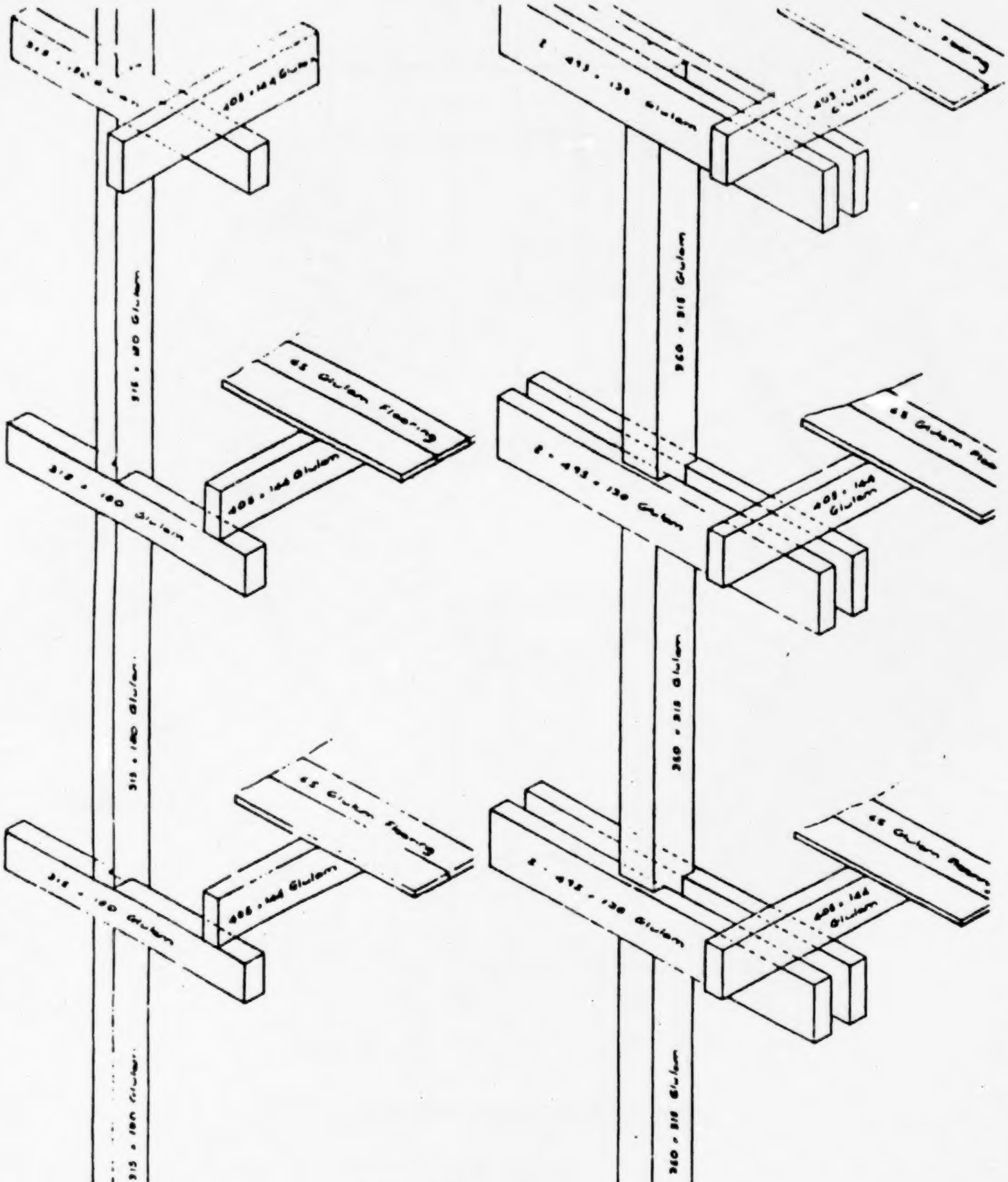
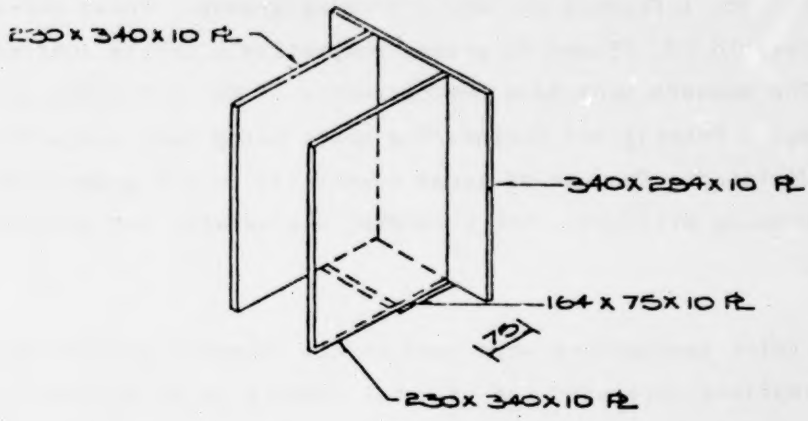

315 x 180 Glulam
405 x 144 Glulam
315 x 180 Glulam
65 Glulam Flooring
315 x 180 Glulam
405 x 144 Glulam
315 x 180 Glulam
65 Glulam Flooring
315 x 180 Glulam
405 x 144 Glulam
315 x 180 Glulam
2 x 495 x 130 Glulam
405 x 144 Glulam
360 x 315 Glulam
2 x 495 x 130 Glulam
405 x 144 Glulam
65 Glulam Flooring
360 x 315 Glulam
2 x 495 x 130 Glulam
405 x 144 Glulam
65 Glulam Flooring
360 x 315 Glulam

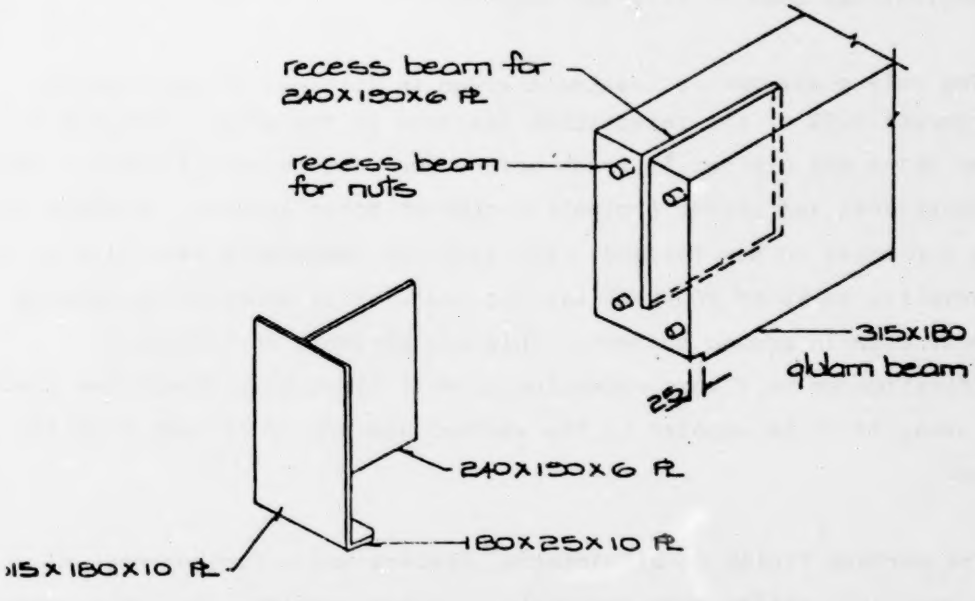
FIG. 11

TYPICAL EXTERNAL COL. TYPICAL INTERNAL COL.


230 X 340 X 10 PL
340 X 284 X 10 PL
164 X 75 X 10 PL
75
230 X 340 X 10 PL

ISOMETRIC OF EXPOSED BRACKET 1:10

FIG. 8


recess beam for 240 X 150 X 6 PL
recess beam for nuts
315 X 180 glulam beam
25
240 X 150 X 6 PL
180 X 25 X 10 PL
15 X 180 X 10 PL

ISOMETRIC OF CONCEALED BRACKET 1:10

FIG. 12

4.4 Specifications

All glulam members were made from untreated radiata pine of Engineering, No. 1 Framing and No. 2 Framing grades. These correspond approximately to F8, F5 and F4 grades respectively in the Australian system. The members were made predominantly of No. 2 Framing grade with a little No. 1 Framing and Engineering grade being used where design stressed dictated. The use of large quantities of low grade timber was possible because stiffness, not strength, was usually the governing criterion.

45 mm thick laminations were used in all internal members while 19 mm thick laminations were used for external members in an attempt to reduce possible splitting due to changes in surface moisture content.

Melamine-urea adhesive was specified for internal members while rescorcinol was used in exterior members.

The only preservative treatment given to internal numbers was to incorporate 0.5% of the insecticide Dieldrin in the water repellant sealer which was applied by brush before the members were erected. This was considered sufficient protection against borer attack; termites not being a problem in New Zealand. For exterior members, a retention of CCA preservative salts of 10 kg/m^3 was required. This provides resistance to decay even in ground contact. This was probably an "overkill" specification as half this retention is sufficient to protect pine timber from decay if it is exposed to the weather and out of contact with the ground.

The surface finish to all internal members was a further coat of water repellant sealer then two coats of gloss varnish. External members were finished with primer, undercoat and finishing coats of paint. Particular attention was paid to the abutting surfaces of the main frame members where an intumescent paint was applied prior to assembly. This was a precaution against fluming should a fire occur.

4.5 Comment on Glulam Costs

In this job the quotations received revealed that various percentage savings could be made as follows:

- Use of untreated timber in lieu of boron treated	6%
- Use of melamine-urea instead of rescorcinol adhesive	8%
- Use of No. 2 Framing in lieu of No. 1 Framing	25%
- Use of No. 1 Framing in lieu of Engineering grade	20%
- Manufacture of large volumes of glulam compared to one-off jobs	50%

5. CONCLUSIONS

Both design and construction expertise in timber is growing in New Zealand to the point where economically competitive buildings can be erected. The vast majority of these are single storey and serving in agricultural or horticultural enterprises but multi storey buildings are also appearing. To achieve economical construction and satisfactory performance it is important that the designer is aware of both the advantages and disadvantages of timber as a structural material.

CASE STUDY OF TIMBER CONSTRUCTION - SOUTH-EAST ASIA

John R. Tadich[1]

TIMBER THE MATERIAL

The use of timber as a structural material in larger buildings requires a greater effort on the behalf of the responsible engineers than would structures using materials such as steel and reinforced concrete having infrastructures which are capable of supplying materials to an engineer's specification. This is not the case with the timber industry. Even in Australia with a timber industry which can be considered to be cohesive by world standards, until recent years could not supply timber to engineering grades.

The lack of material standards for timber which in some developing countries extends beyond those required for durability and strength to the very basic property of sizes.

I do not wish to dwell on this aspect as I am sure this has been adequately discussed in earlier papers, except to say that before attempting to design timber structures it is important to determine the possible sources of the material and ascertain the following details:

(a) Finished timber sizes and corresponding lengths.
(b) Quantities of relevant sizes.
(c) Species.
(d) Availability of quality control and grading standards and ability of suppliers to conform.

On many occasions engineers have had to redesign timber structures because the local timber industry could not supply to specification. Avoid using wherever possible, large sections of high grade material. Design structures to utilise common sizes and grades. Remember the larger the section the more difficult it is to achieve the higher grades.

---

[1] Technical Director, Gang-Nail Australia Ltd., Mulgrave, Victoria, Australia.

DESIGN CONCEPT

Before attempting to conceive the structure of the building, the engineer should also determine the skills and infrastructure available in fabrication of components and in the erection of the components. Fundamental decisions such as the type of connectors available influence the conceptual design of timber buildings.

Factors to consider when choosing connector types -

(a) Can Components be Pre-Fabricated?

If there is a timber pre-fabrication plant within economical transportation distance, many design and fabrication problems can be easily resolved.

There are many companies now being established in developing nations which are specialists in the manufacture of pre-fabricated structural components for buildings.

These plants invariably use multi-tooth connector systems which have the backing of specialist timber engineering services provided by the manufacturers of the multi-tooth connectors. Before getting involved in detailed design, contact with one or two of these companies would be invaluable. Most would be able to prepare a feasibility design and costing on the project. Even though there may not be a pre-fabrication plant in the immediate area, do not discount pre-fabrication as on many projects it has been proven economical to transport components many hundreds of kilometres. On larger scale projects pre-fabricators have equipment which is transportable and may be established on the job site.

(b) Architectural Design

Architectural design sometimes will dictate the type of connector to be used. For example, in some public buildings where the structure is to be an architectural feature of the building, it may require a large timber section to be used and connectors such as split rings or shear plates.

(c) Cost of Connectors

The cost of connectors and the installation of connectors is a very significant cost in timber structures. The in place cost of connectors can vary from 10% to 30% of the cost of the building element, therefore, careful analysis of the cost of each type is essential before detailed design. The choice of connector can also influence the volume of timber used in a structure. For example, the use of double top or bottom chords in a truss may be necessary to avoid large eccentrically loaded joints, whereas a single member may have been adequate.

(d) Environment

Service environment, if corrosive, may eliminate many types of connectors, e.g. common steel nails which will quickly corrode and fail. There are several connectors which are available which offer very good corrosion protection.

(i) Ceramic connectors which are shear connectors manufactured from steatite and if used with brass or stainless steel bolts, provide good protection. These connectors have been used extensively in cooling tower construction in large air-conditioning plants.

(ii) Stainless steel multi-tooth connectors are available on special request through some manufacturers. These connectors are obviously much more expensive than the standard galvanised connector and should only be used in extreme circumstances. The galvanised connector would provide satisfactory service in most conditions.

(e) Connector Installation Cost

As the cost of connector installation is a major cost in fabricating building components, it influences greatly the layout of the building and, hence, the economy beyond that of the fabrication of the element.

If installation cost is small and it is possible to design lightweight elements, these elements can be placed at very close centres and, hence, reduce the cost of secondary structural members dramatically.

For example, multi-tooth trusses supporting large span heavyweight tiled roofing with a ceiling at bottom chord level, would be placed at 600 mm or 900 mm centres using standard battens fixed directly to chords. The alternative would be to use bolted or split-ring trusses at 1800 mm or 2000 mm centres with under purlins and intermediate rafters and ceiling joists which adds considerably to material and on-site labour costs.

IMPORTANT FEATURES OF CONNECTORS

As connector performance has been covered in an earlier paper, I do not propose to reiterate this information, but to outline pitfalls in the use of some connectors.

(a) Split Rings

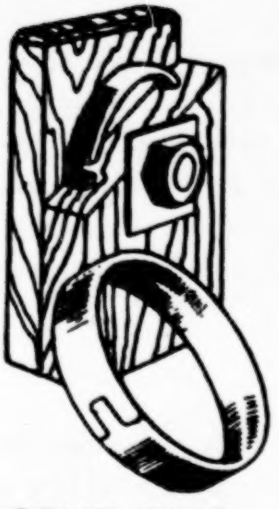
Split rings are so named because they are in fact rings which have a split (see Figure 1). The function of the split is to accommodate timber shrinkage. Therefore, in the installation the grooves should be formed so that the connector is expanded when placed into the groove, sufficiently to prevent the ring completely closing when the timber has shrunk.

A common practice amongst some contractors is to use sections of pipe in lieu of split rings. This will of course restrain timber shrinkage and induce splitting which could cause major problems.

This practice also increases the joint slipping as the parallel sides of pipe do not take up the groove tolerance as does the tapered section of the split ring.

(b) Multi-Connector Joints

Extreme care should be taken to avoid joint details which restrain timber shrinkage, e.g. using bolts spaced across the grain is inviting disaster (see Figure 2).



SPLIT RING

FIG 1

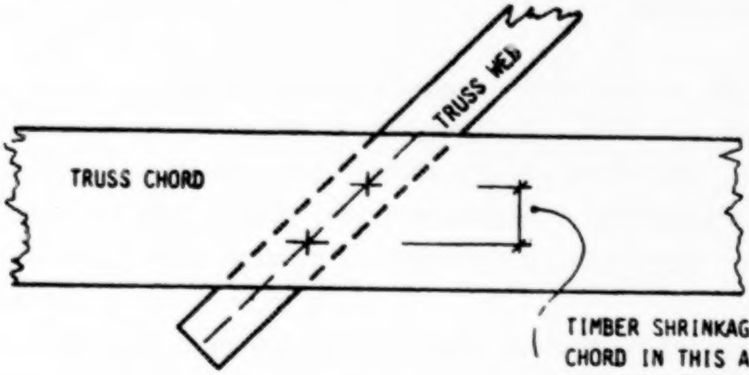
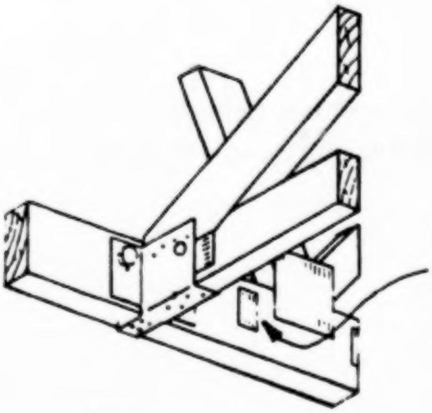

TRUSS WEB
TRUSS CHORD
TIMBER SHRINKAGE OF CHORD IN THIS AREA WILL CAUSE SPLITTING.

FIG 2

AS TIMBER SHRINKS IN SIZE ACROSS THE GRAIN AND NOT LONGITUDINALLY WHEN TIMBER DRIES INTERNAL STRESSES ARE INDUCED WHICH CAN CAUSE SEVERE SPLITTING.



MULTI-TOOTH CONNECTOR TO PREVENT SPLITTING DUE TO PRESENCE OF TENSION PERPENDICULAR TO GRAIN.

FIG 3

(c) Load Perpendicular to Grain of Timber

Connections should never be designed so as to develop tension perpendicular to the grain without some precaution against splitting occurring. A common case is the support of standard trusses by girder trusses in complex roof designs.

The girder bracket connection here is always reinforced by a multi-tooth connector placed either side of the bracket to distribute tension perpendicular forces across the face of the girder bottom chord.(see Figure 3).

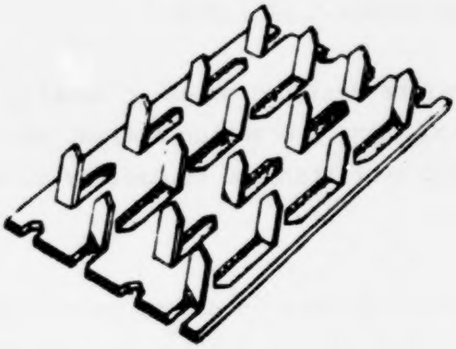
(d) Installation of Multi-Tooth Connectors

Most multi-tooth connectors are designed to be pressed into the timber using substantial hydraulic or impact presses. These connectors are readily recognisable as the teeth protrude perpendicular to the plate (see Figure 4). It is possible to drive this type into soft timber on site with special hammers, however, they may not be capable of developing the published design loads. Therefore this practice should be avoided.

However, there are multi-tooth connectors that are designed to be installed with a hammer, e.g. Nail-on-Plates and Tylok Connectors (see Figure 5).

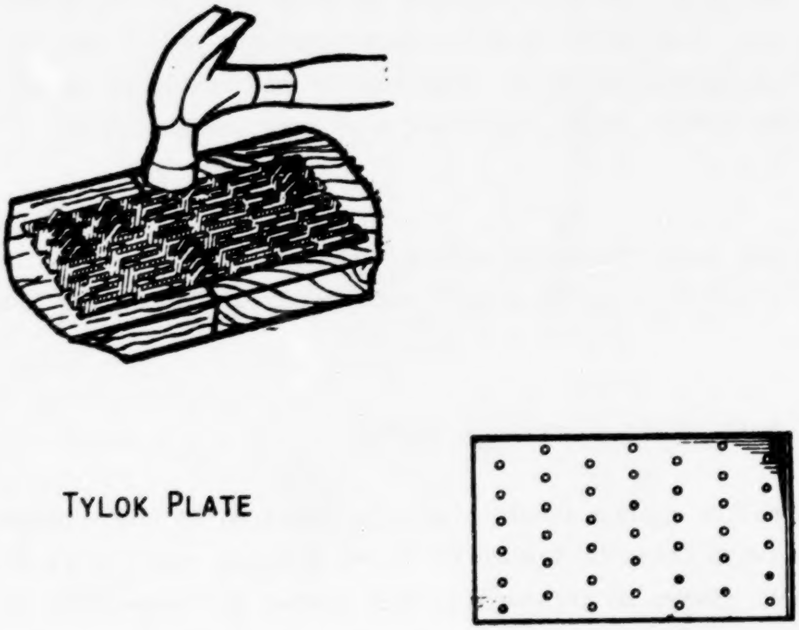
(e) Design of Multi-Tooth Connectors/Joints

Multi-tooth connector joints should always be designed so that compression loads are always taken by timber to timber bearing (see Figure 6). The joints should always be arranged so that timber shrinkage will not cause compression loads to be shed to connectors (see Figure 7) and possibly cause buckling of plates and significant reduction in strength.



FACTORY FIXED MULTI-TOOTHED CONNECTOR

FIG 4

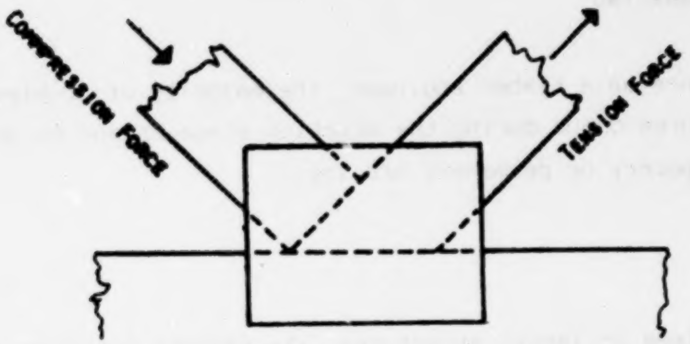


TYLOK PLATE

TECO NAIL-ON-PLATE

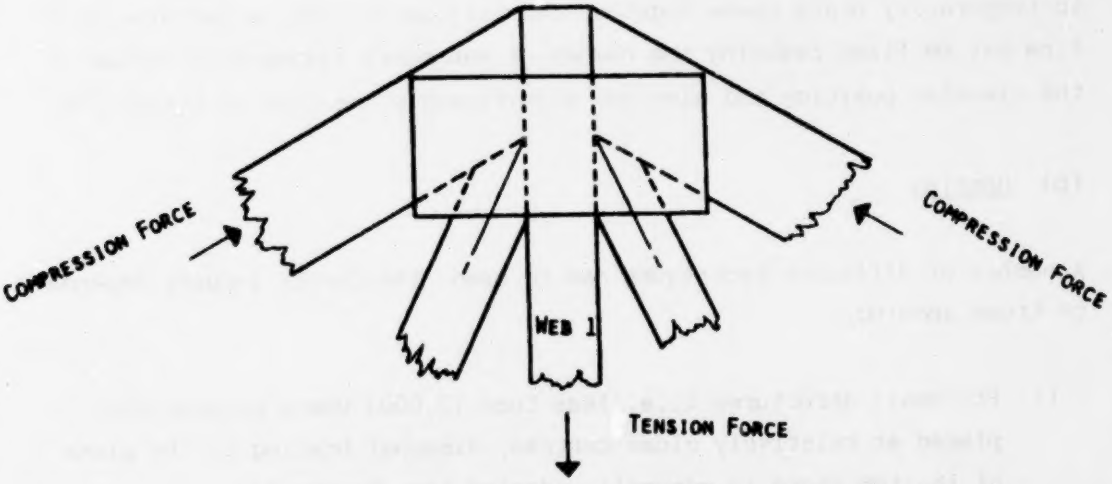
SITE FIXED MULTI-TOOTH CONNECTORS

FIG 5.


COMPRESSION FORCE
TENSION FORCE

TYPICAL JOINT ARRANGEMENT TO RESTRAIN COMPRESSION LOAD VIA END BEARING.

FIG 6


COMPRESSION FORCE
COMPRESSION FORCE
WEB 1
TENSION FORCE

SHRINKAGE OF WEB 1 WILL CAUSE BUCKLING OF MULTI-TOOTH CONNECTOR AND WILL REDUCE STRENGTH OF JOINT.

FIG 7

ERECTION AND BRACING

In my experience as a timber engineer, the majority of problems in timber structures occur during the erection stage or due to poorly installed temporary or permanent bracing.

(a) Erection

Erection problems on larger structures like factory buildings seem to evolve around the lack of expertise in erection of timber structures. The rigging crews around have had plenty of experience with steel and tend to handle timber in the same manner. Large span timber members are much more flexible than similar steel members and hence, require considerably more care. Our company has adopted the policy of always detailing erection procedures as part of the design documents. These instructions include:

(i) Instructions on lifting
(ii) Temporary bracing.

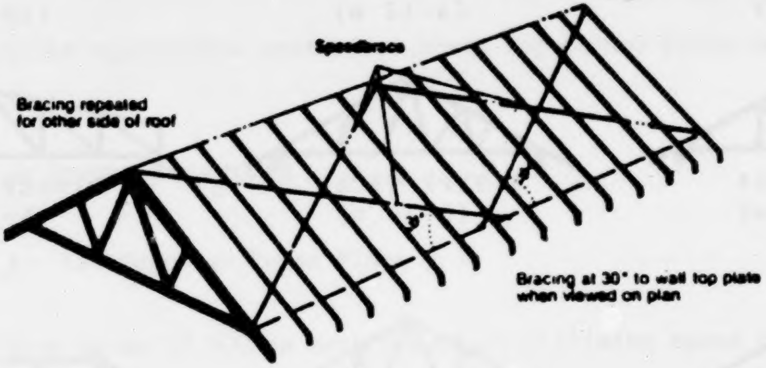
The usual technique is to stand up groups of trusses on the ground and to temporarily brace these together and lift as a group. Alternate purlins may be fixed reducing the number of man hours fixing ancillaries in the elevated position and also cut significantly the cost of crane hire.

(b) Bracing

A number of different techniques can be used, the choice largely depends on truss spacing.

(i) For small structures (i.e. less than 13,000) where trusses are placed at relatively close centres, diagonal bracing in the plane of the top chord is generally adopted (see Figure 8).

(ii) For large spans and wider spaced trusses, special pre-fabricated bracing trusses or on-site bracing bays are used (see Appendix B). On long buildings it is good practice to include intermediate braced panels, even though the panels at each end may carry the required load. As with long buildings, it is possible to have sufficient slip at joints of members which tie each end together to allow large buckles to develop between braced bays.


Ridge
Speedbrace
Bracing repeated for other side of roof
30°
Bracing at 30° to wall top plate when viewed on plan

TYPICAL BRACING FOR CLOSELY SPACED TRUSSES UP TO 13M SPAN

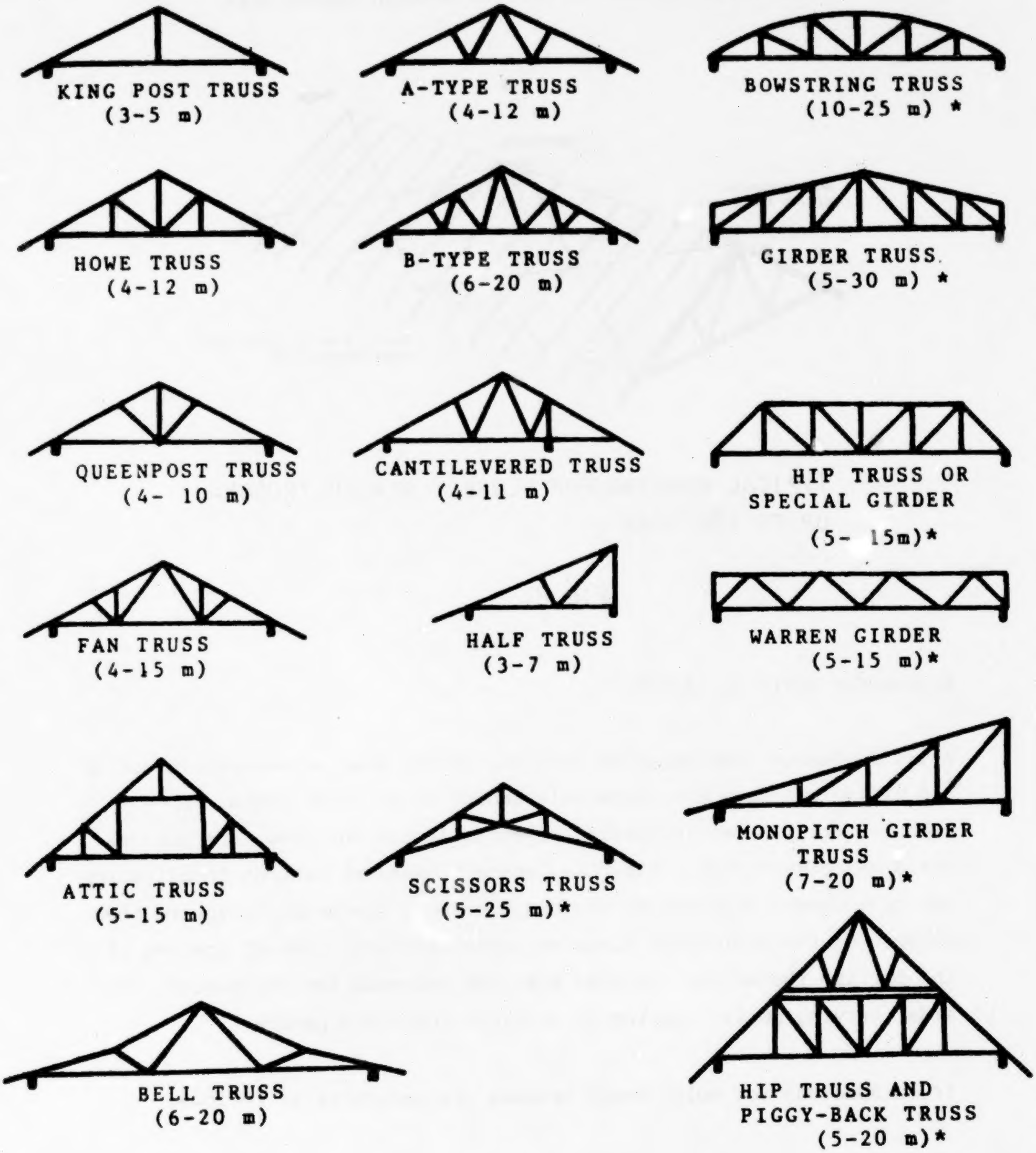
FIG 8.

ECONOMICAL BUILDING LAYOUT

A little design time spent on determining the most economical layout of a building can save a considerable amount of building costs. For example, for an open industrial or agricultural building, the purlins are a major cost item. Therefore, space trusses at centres to allow the use of a common size and at its maximum span, preferably allowing them to span continuously over three supports. In turn, look at spacing of the purlins themselves, as wide spacings increase top chord size. Consider varying purlin spacing in critical top chord panels.

Truss spacings for multi-tooth trusses are generally as follows:

Roofing	Ceiling	Spacing
Steel sheet	Nil	2000 - 4000
CAC sheet	Nil	2000 - 3600
Steel sheet	Yes	900 - 1800
CAC sheet	Yes	900 - 1800
Concrete tiles	Yes	600 - 1200


KING POST TRUSS
(3-5 m)
A-TYPE TRUSS
(4-12 m)
BOWSTRING TRUSS
(10-25 m) *
HOWE TRUSS
(4-12 m)
B-TYPE TRUSS
(6-20 m)
GIRDER TRUSS
(5-30 m) *
QUEENPOST TRUSS
(4- 10 m)
CANTILEVERED TRUSS
(4-11 m)
HIP TRUSS OR
SPECIAL GIRDER
(5- 15m)*
FAN TRUSS
(4-15 m)
HALF TRUSS
(3-7 m)
WARREN GIRDER
(5-15 m)*
ATTIC TRUSS
(5-15 m)
SCISSORS TRUSS
(5-25 m)*
MONOPITCH GIRDER
TRUSS
(7-20 m)*
BELL TRUSS
(6-20 m)
HIP TRUSS AND
PIGGY-BACK TRUSS
(5-20 m)*

Common truss configurations
and their approximate span range.

* Number of bays can be varied
according to truss span.

FIGURE 9

As the labour cost in the manufacture of multi-tooth trusses is minimal, compared to other types, it is not economical to use large members and space the trusses at wide centres. It is more economical to use two trusses at closer centres and reduce ancilliary member sizes.

Figure 9 gives approximate economical spans for common truss shapes and spacings.

EXAMPLES OF TIMBER STRUCTURES

Appendix A - Lee Sang Lon Truss Plant

This building is an 18,000 mm arch structure utilising space columns.

NOTES

(a) Very small timber sizes 125 x 50 top and bottom chords.

(b) 'T' stiffener or bottom chord to give additional lateral stiffness.

(c) Staggered bolts at footing to column connection to reduce tendency to split. Possibly Gang-Nail connector should have been used here to prevent splitting.

(d) Pole type R.C. footing.

Appendix B - Open shed at Temerloch

This building is a 24,000 x 73,000 mm open building. Trusses and columns have been designed at rigid element to restrain lateral movement.

NOTE:

(a) Spacing of braced bays.

(b) Use of wind truss to restrain truss between column from lateral movement.

(c) Truss manufactured in two sections and spliced on site using timber splice plates and bolts.

(d) Minimal use of steel brackets.

(e) Connection of T1 to GT40 major load taken by bearing. Bolts used for lateral restraint GT40 top and bottom chord.

Appendix C - Malaysian Community Developement - Subang Jaya

1. Typical low-rise housing project.

NOTE:

(a) Close spacing of 600 mm.

(b) Top chords act as rafters with tile battens fixed directly. Bottom chords as ceiling joists with A.C. sheet fixed direct.

(c) Top chord bracing is fixed in place of top chord to restrain top chord from lateral buckling.

(d) Vertical cross bracing over internal supports.

2. Typical shop-houses

Small complex structures such as this bell shape roof can be economical using pre-fabricated timber trusses. Because -

(a) Production techniques

(b) Computer systems employed to automatically produce structural design and fabrication details

projects such as this community development can, through local fabrication plants or directly through manufacturers of multi-tooth connectors, assist using their computer system with structural designs.

Appendix D - Straits Timber Products Factory

This structure is an illustration of how timber can be used very successfully in conjunction with other building materials, in this case reinforced concrete, to achieve maximum economy and serviceability. This building used R.C. columns and perimeter beams, with 18,000 clear span timber trusses. Note the close spacing of 1800 mm for large span trusses.

This building was so successful that this first stage was repeated three times.

Appendix E - French Trade Building - Kuala Lumpur

This structure is a relatively complex building requiring large open areas. The building was originally intended to be a temporary building so the cost of construction was very critical. It is also a good example of application of a parallel chord truss.

Appendix F - Typical erection and bracing details for large span, closely spaced trusses.

STRESS GRADES AND TIMBER CONSTRUCTION ECONOMIES, EXEMPLIFIED BY THE UNIDO PREFABRICATED TIMBER BRIDGE

C. R. Francis[1]

The UNIDO modular prefabricated bridge was designed in Kenya in 1973 by James E. Collins as a forest access bridge. From 1975 to 1977 a UNIDO project allowed him to develop the design. The system now covers stress grades from F4 to F34 and 12 different design loadings. The design was done in accordance with AS1720. A comprehensive manual has been published by UNIDO (DP/ID/SER.A/201) in English and French.

The bridge consists of a nail-laminated deck platform supported on 45° Warren girders. The Warren girders may number from two to ten across the bridge, and the span may reach 30m. The girders are composed of 3m long truss units joined by steel spigot and socket end pieces and mild steel pinned bottom chords.

The bridge is designed for prefabrication in a workshop and site assembly using only hammer and nails for major components. A launching system using two derricks and a cable way has been devised. No major plant except a mobile welder is required for assembly, but an electric generator and portable power tools speed the work considerably.

The writer inherited the bridge project as part of an expanded project in Kenya in 1979, and in 1981 he made brief visits to Honduras and to Madagascar to advise on the initiation of bridge projects there, using the UNIDO system.

In both countries it was required to advise on workshop, manpower and material requirements for the desired programme. The work fell logically into the following order:

1. Ascertain the required bridging programme and design loadings

2. Determine timber characteristics and availability

---

[1] Timber Engineer (at the time Chief Technical Adviser of UNIDO Project DP/SRL/79/053 - Research and Development for the Utilization of Rubberwood and Coconut Wood).

3. Determine the annual requirements of trusses, hence timber and steel quantities

4. Determine necessary facilities for fabrication and erection.

The key item in this series is No. 2. The timber strength and quality largely determine the magnitude of items 4 and 5 and may also feed back to item 1. Timber strength - "F" rating is of major importance, but so also are presentation and availability.

Fig. 1 has been prepared from the tables contained in the UNIDO report referred to above, for HS20 vehicle loading. Examination of this chart reveals three interesting facts:

1. The weakest timbers, say up to F7 have only about half the span capacity of the strongest, with an absolute span limit of 15-18m

2. For a particular span about twice as many trusses are needed with weak timber than with strong timbers

3. No major advantage is gained by using timber stronger than F11

For economic construction, the stress grading problem simplifies to finding a source of timber of not weaker than F11, or as second best not weaker than F7. More precise identification is not really required. There is no advantage gained with this design in insisting on, for example F17 which could well be the case if some old fashioned hardwood specifications which the writer has seen were to be followed.

Presentation

The two countries visited, Honduras and Madagascar are at opposite ends of timber industrz sophistication in developing countries. For years Honduras has supplied "Pitch pine" (P. carribea and P. oocarta) to the southern U.S. market. Export oriented mills are capable of presenting timber to high standards of dimensional tolerances and grade uniformity to U.S. commercial standards. In Madagascar, timber is available only in 4m and 6m lengths, roughly hand hewn, and if hardwood, described simply as "bois du forêt" with no species separation or grading.

The writer was most impressed by the Honduras pine. Large perfectly clear timbers were common. Test results and application of U.S. Southern pine grade stresses indicated at least F14, more probably F17 for "Structural" grades. However the timber was available only in U.S. sizes where the nominal thickness of scantlings are 1/2" under nominal size. The designs for the bridge call for 55 mm thick (2") timber. Some brief calculations indicated that F11 stresses both in bending and joint details should not be exceeded with scant "F14 plus" timber provided the "dense" U.S. qualification was complied with.

The problems in Madagascar were much greater. Examination of the available timber showed that the bulk was a dense red eucalyptus species, witha small quantity of lower density yellowish timber. The eucalyptus was most likely E. robusta and flotation experiments in a tumbler of water showed densities of around 0.9. It was generally clear and straight grained, so with some on-site selection, it could easily qualify as F17. However, the dimensions available were narrower and thicker than those called for in the design. The only major woodworking machine available was a large overhand planer and with this it was possible to produce timber of 6 - 7 cm thick x 20 - 23 cm wide.

The design calls for nail laminated diagonals and king posts, and with only 10 cm nails available it was considered that there would be insufficient point penetration. Also very major efforts would bve required to reduce the timber to 5 cm thickness.

The trusses were redesigned on the basis of a single 6.5 cm diagonal and king post. Again with these size and fastening length reductions it was decided that F11 design tables would be adequate for a H20 load.

It is very difficult to provide cost comparisons between different countries. In Kenya a preliminary estimate would be about KShg 4400 per metre of 4-girder bridge. No cost estimates were done by the writer in Honduras or Madagascar. In Madagascar a peak truss production rate of 8 per day was reached with a 10 man gang, but including other work an average of 6 would be more reasonable. In Honduras planning was done on the basis of 4 trusses per day with 6 men, but the writer does not know what was the subsequent experience.

The two important lessons to be taken from this bridging experience are :

1. Labour and material quantities can be quite dramatically reduced by relatively small jumps in the "F" range

2. Departures from specified sizes may necessitate effective downgrading of timber to achieve a required structural strength

Relationship between cost and strength

The preceeding discussion has been on a predetermined complete design. Given the data of Fig. 1 or similar for other design loads a complete estimate can be made for any timber stress grade. This section will explore a more general relationship of strength and cost.

The engineering design process in timber generally follows the sequence:

1. Architectural requirement of span, spacing, loading

2. Selection of structural solution - beam, truss, arch

3. Selection of material type - scantling, light glulam, heavy glulam

4. Calculation of necessary sections

5. Detailing

At the end of step 4, preliminary estimates may be made and steps 2, 3 and 4 may be repeated in various systems and materials.

In timber design, with its largely rectangular sections, step 3 is a critical one, since selection of material type largely fixes the breadth of the sections due to the limited commercial range of timber sizes.

In the design of a beam, using conventional engineering symbols,

$$f = \frac{M}{Z} = \frac{6M}{bd^2} = \frac{K}{d^2}$$

$$d = \frac{K^2}{\sqrt{f}} \quad \text{—} \quad (1)$$

Timber is sold by volume. In two different solutions to the same problem, to have the same cost :

Cost $PbdL$ where P price

$$P_1bd_1L = P_2bd_2L \_\_\_\_\_\_ (2)$$

Note that this not absolute, since b is only more or less fixed at design stage (2) but it is certainly more constant than d, which is what we find in design stage (4).

Substituting equation (1) in equation (2)

$$P_1 b \frac{K_2}{\sqrt{F}_1} L = P_2 b \frac{K_2}{\sqrt{F_2}} L$$

Cancelling and rearranging,

$$\frac{P_1}{P_2} = \sqrt{\frac{F_1}{F_2}} \quad \text{——} \quad (3)$$

A very large number of beam designs are limited by deflection considerations, so that the deflection to span ratio does not exceed a standard value. In this case, identical reasoning using $I = \frac{bd^3}{12}$

leads to $$\frac{P_1}{P_2} = \sqrt[3]{\frac{E_1}{E_2}} \quad \text{——} \quad (4)$$

Equation (4) also covers the case of laterally restrained columns which are designed to an Euler type formula, such as truss top chords.

Given a price list of timbers of various species and grades, and a table of design stresses, a designer can rank various timbers as good or bad structural buys, at least in general terms, by working between the limits set by equations (3) and (4) and bearing in mind the assumptions made in their derivation.

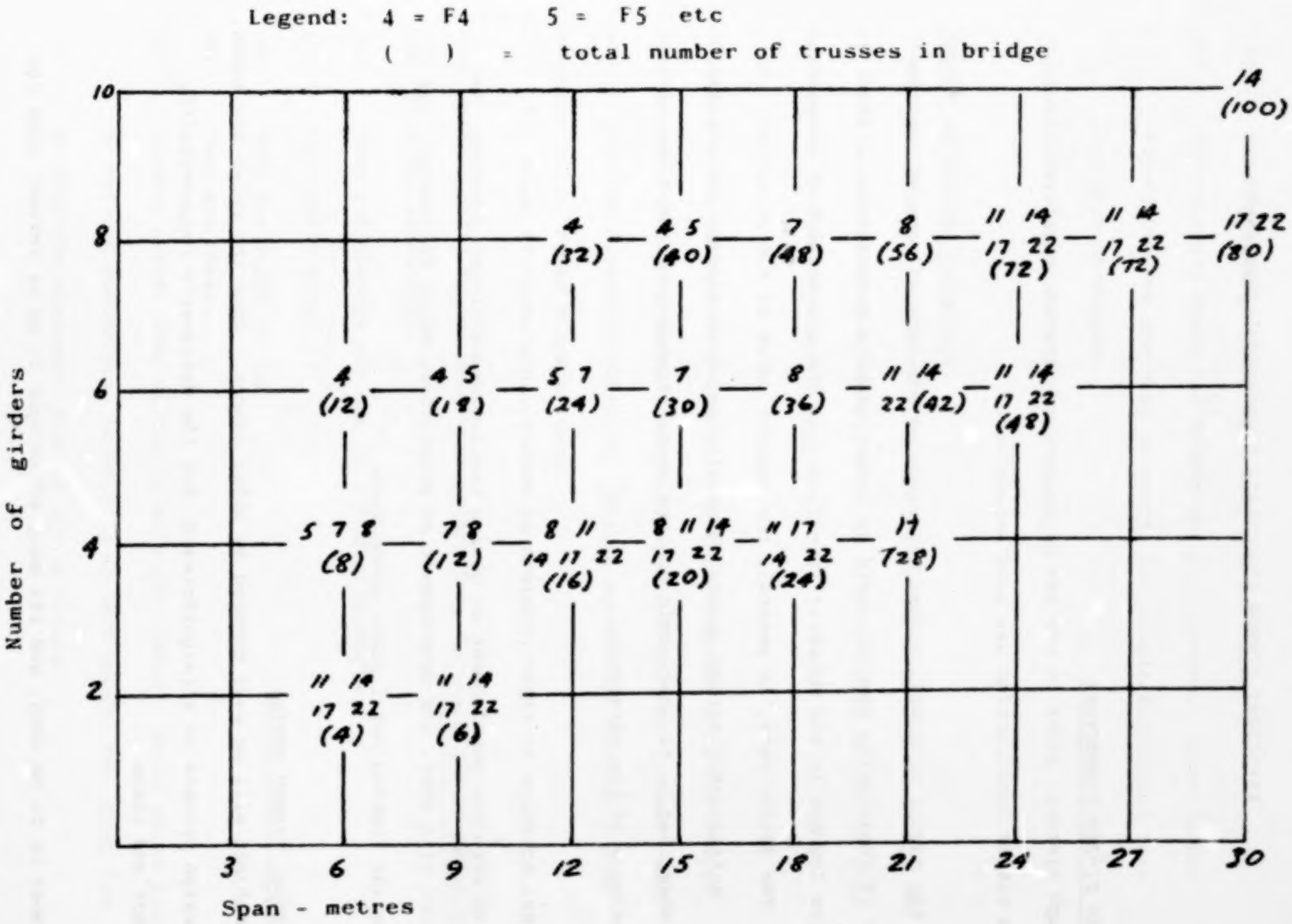


FIG 1.

Bridge Construction - HS20 Design Load

EFFICIENT TIMBER STRUCTURES USING METAL CONNECTORS

E. E. Dagley[1]

MODERN TIMBER CONNECTORS

Through history, prior to the use of welded and riveted structural steel, heavy timber construction was used extensively.

With the advent of steel, timber was replaced for several reasons, not the least of these being that it could no longer compete economically. The problem lay not in the material itself, but in the old methods of connecting. The joints were, in general, the weakest parts of the structure. To make sufficiently strong joints generally required lapping and overlays of timber leading to uneconomic use and under-stressing of the timber in the balance of the structure.

The real solution to these problems is recent, only about 25 years old. It came with the development of gluing techniques and finger jointing, and parallel with that, the development of proprietory metal fastenings, and in particular toothed metal plate connectors.

STRUCTURAL TIMBER DESIGN

This subject will be well covered by other papers. Once the rules are known, the design process is straightforward, but the designer's responsibility does not end there.

If timber is to be used, and its many advantages to be maximised, then the following factors must be allowed for at the design stage.

---

[1] Managing Director, Gang-Nail N.Z. Ltd., Auckland, New Zealand.

(a) Site fastenings which are cheap and efficient.

(b) Structurally sound but simple bracing systems. Large timber structures are not easy to brace, but simple techniques have now been developed.

(c) Safe, quick and efficient erection.

MODERN MECHANICAL FASTENERS

In earlier decades fastening of timber was generally limited to the use of dowells, bolts, screws, nails, etc. or alternatively relied on such methods as dovetailing, morticing, etc.

The resultant joints were comparatively weak compared with the design strength of the timber member, were subject to slip, and in some cases were a slow and expensive exercise. Modern mechanical fasteners have overcome all of these disadvantages.

They are generally highly developed proprietory fasteners which exhibit the following advantages:

(a) They efficiently use the full timber strength.

(b) They don't slip.

(c) They are quick to use.

(d) They are cheap.

(e) In many cases, they virtually weld the timber. Using tooth plate or Nail-on connectors, especially in soft woods, the joint can be made the strongest part of the structure.

BUILDING REQUIREMENTS

The designer must allow for many detailed requirements, but the main ones can perhaps be summarised under the following headings:

- (a) It must have structural integrity. It must be capable of withstanding all possible load combinations.
- (b) It should be capable of completion as economically as possible. In a timber framed building the designer should ensure that he is efficiently utilising the timber, the fastenings, labour, etc.
- (c) The building should be as aesthetically pleasing as possible. Architectural features and design should be adopted to ensure an attractive result.
- (d) It should be durable, in keeping with the planned life of the structure.

Many slides will be shown to illustrate what has been done in New Zealand over the last decade. These slides will illustrate the above, but in addition, will show how mechanically fastened timber structures are

- (a) Versatile and attractive
- (b) Economical
- (c) Easy to erect
- (d) Structurally superior
- (e) Good fire performance (compared with fire resistance)
- (f) Corrosion resistant
- (g) Energy efficient
- (h) Easily interfaced with other materials such as steel and concrete, etc.

SITE FIXINGS

In New Zealand heavy structural Nail-on techniques have been developed, which have allowed us to supply and erect very large timber structures. Nail-on plate is punched in 1.2mm, 2.0mm, 3.0mm and 5,0mm thicknesses. This can be cut and fabricated to make a very wide variety of brackets and fixings. It is used at the knee and apex of portal frames, for connecting to other structural materials such as steel or concrete, and is also used for the site splicing of large truss components.

BRACING

We have found it more than just desirable to fully brace the roof components on the ground prior to erection. To achieve this, we use Gang-Nail connected panel or "K" braces so that truss groups are fully assembled and braced before erection.

The roof diaphragm is then often braced using comparatively light metal strap which we call Strip Brace, tensioned with a convenient high-speed adjustable clip device. As the Strip Brace has a capacity loading of only about 8 kN, a number of parallel strips are often used to develop the bracing loads required. We have found this to be the quickest and most economical method.

ERECTION

The trusses are spaced on the ground to their relative final positions. They are then braced in groups of between two and six trusses and while still on the ground the final bracing is fully installed together with the purlins, the roof services and walkways and often also the roof sheathing.

The Nail-on brackets or stirrups at the top of the columns are already in place and one or more cranes are then utilised to lift the roof sections.

Trusses under 12.0m free span in general do not require cranes, and, provided experienced workmen are on site, they can be lifted into place singly and often by hand. No mechanical lifting is required for small trusses.

Trusses over 15.0m free span we have found become an engineering exercise, and over 20.0m free span, the above-described techniques must be used or equivalent methods to avoid the real danger of a collapse during erection.

GANGLAM

This technique comprises building up very large timber beams and other components using tooth plate connectors.

Shorter and smaller members are first end to end laminated to obtain the required lengths. The timber used is generally 50mm thick and the longer members are then edge laminated up to any required width, but commonly 600mm and more.

These 50mm thick "planks" are then "slabbed" together to produce the required design thickness for the composite beam, commonly up to 200mm and more.

The "planks" are commonly nailed together where interface shear requirements are low, such as for a simple beam. However, interface shear requirements can be higher as in the case of a column or can be extremely high as in the case of a laminated girder.

In these cases, heavy nailing or bolting is required. Where very heavy compression loads are to be carried, special Nail-on members are designed or alternatively edge trusses or other means are used to hold the member straight.

CURVED ARCHES

A special form of Ganglam has been introduced in New Zealand during the last nine months. Thin laminates, usually 50 x 25mm are curved to the required shape, and the required number laid up, face to face. Tooth plate connectors are then driven into the edge faces to carry the interface shear between laminates and the arch then retains its shape when released, and if correctly designed, is capable of withstanding the necessary design loads.

TIMBER CONSTRUCTION IN DEVELOPING COUNTRIES

C. R. Francis[1]

My personal experience of developing countries is limited to five and they range widely in location, climate and history. However they all have rather similar approaches and problems in timber utilisation for structural purposes.

1. Degree of Utilisation. Generally timber is confined to roof framing, sometimes first floor beams in "quality" housing or else is confined to shantys. Roof framing is heavy and widely spaced;lightweight trussed rafters are unknown. There may be some use of standard prefabricated buildings or components. There is no use of component systems e.g. trussed rafters predesigned and prefabricated to individual architectural requirements, or standard-spaced stud framed wall sections. This is due to:

1. Non-availability of accurately dimensioned timber
2. Ignorance of the system, approach or inability to adapt a system to local circumstances
3. Lack of, or inappropriate design codes. Examples of this include insistance on the use of European loadings complete with snow loads for tropical roof design. In another country I was told that stud framed walls could never be allowed since the load of the studs on the bottom plate exceeded the (imported) mid 1920's German allowable timber stresses.

2. Preservation. Lack of preservation severely limits availability of timber. The general pattern, which includes experience in New Zealand and Australia is that there has been an abundance of durable species which are now exhausted, or rapidly becoming so.

In NZ and Australia the shortfall was made good by preservation of non-durable timber - notably Radiata pine. The project I am currently working on is completely based on boron treatment of otherwise useless rubber wood which is extremely prone to Lyctus attack. Some interesting statistics from one major wood preserving group are :

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COUNTRY	NO. OF PRESSURE PLANTS	POPULATION (MILLIONS)	POPULATION PER PLANT (MILLIONS)
Great Britain	192	55.9	.29
New Zealand	124	3.1	.025
South Pacific	12	0.5	.041
Singapore, Sri Lanka, Indonesia, Philippines, Taiwan, Thailand	54	204.1	3.8
Malaysia	87	12.3	.1414
India	150	605.8	4
Turkey	3	39.2	13
Scandanvia	46	22.1	.48
Australia	114	13.5	.1184
Papua New Guinea	3	2.8	.933
Canada	20	22.9	1.14

Some of the figures are of doubtful interpretation, but they do show a vast imbalance between industrialised and developing countries. Also, this is only one company, albeit a major one. The figures show clearly the extremes between an intensely preservation conscious country like N.Z. and third world countries with millions of people per preservation plant.

It is my firm belief that sound preservation practices are the most important single factor in establishment and acceptance of timber construction. The major inhibiting factor does not appear to be the direct cost of the plant or chemicals, or lack of skill to run the plant. Rather it is shortage of working capital to cover the cost of drying ready for pressure treatment, or of diffusion treatment storage time. Shortage of working capital similarly inhibits drying, not only for structural work but also for furniture and joinery.

Skills, Training, Tools. All the wood using developing countries appear capable of good standards of craftmanship in furniture and joinery using hand tools and simple machines. Use of portable power tools and four-side planers is rare. General availability of accurately surfaced timber to standard dimensions is almost unknown. Practices with machines are extremely dangerous. Saws are used without guards or riving knives, planers and spindle moulders without guards. In one Marxist oriented country after inspecting a woodworking factory I created considerable embarrassment when I commented that in my own capitalist country the unions would not permit their members to work in such a dangerous place.

A major problem is teaching illiterate workmen to measure. In these circumstances wide use should be made of gauges and templates.

For construction work the most useful tools are the radial arm saw and the portable circular saw. Used intelligently, these can speed up work and also improve over hand tool precision. However instruction in their safe and efficient use is essential.

In promoting use of structural timber my thoughts on the order of importance of things to be done are :

1. Organise a sound system of preservative treatment with provision for enforcement of standards.

2. Standardise dimensions and tolerances and refuse to purchase timber which does not conform to these.

3. Adopt a system of grading and an appropriate timber design code. AS1720 is an excellent flexible code which can be used in tropical countries.

4. Concentrate at first on simple bulk products e.g. stud frame walls or a trussed rafter system to demonstrate economical high quality timber construction.